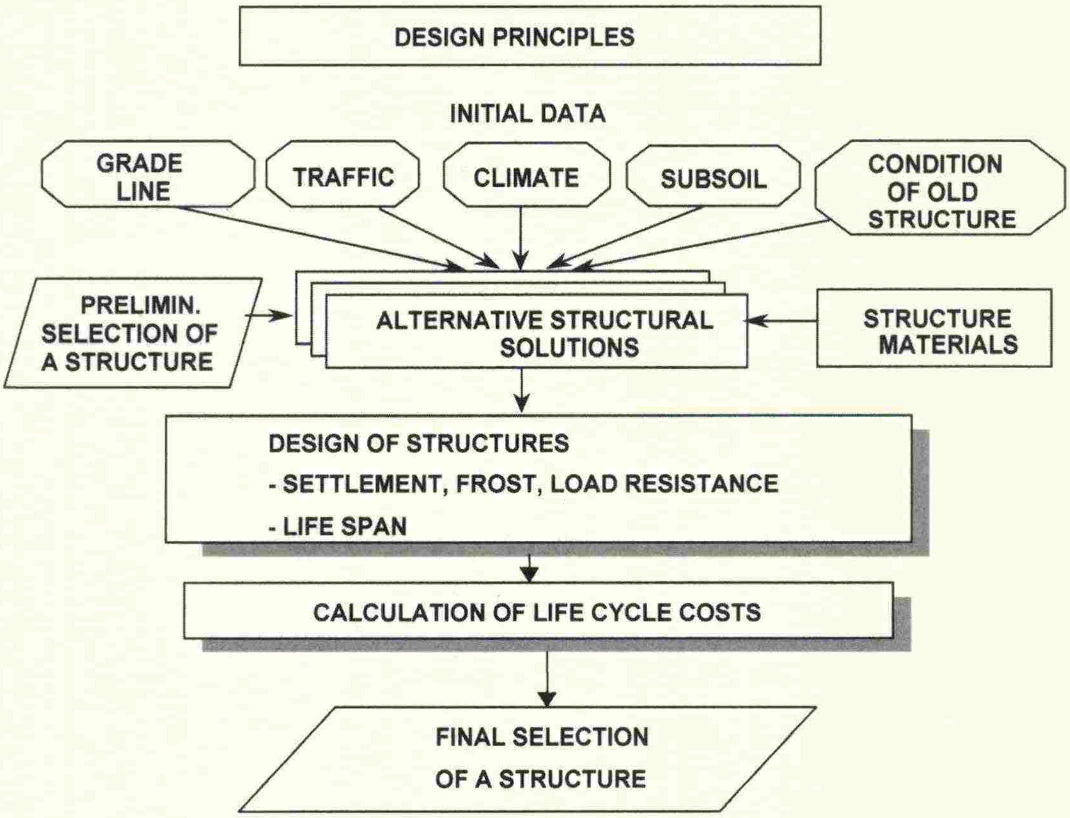
FINNISH ROAD ADMINISTRATION

Markku Tammirinne, Aarno Valkeisenmäki and Esko Ehrola
(Editors)

Road Structures Research Programme 1994 - 2001

Summary report

Finnra Reports 37/2002



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ABSTRACT

The Road Structures Research Programme (S4) was carried out in 1994-2001 by the Finnish Road Administration. The main part of the program consisted of the Road Structures Research Project (TPPT project) consisting development of pavement and the substructures. The objective of the results obtained from the TPPT project is to be able to construct new roads and repair old paved roads more durable while reducing the annual cost of the road. The goal of the TPPT project was to develop the design methods of the pavements to make them more durable and to make it possible to estimate their performance during their entire life span. If the performance of the road structures can be estimated in advance, the risk of unexpected damages decreases.

The results of the TPPT project were compiled into a design system. The design guidelines and descriptions of the design system present the procedures and methods with which a pavement for a particular site can be designed to keep settlement, frost damage and load-bearing capacity under control. The TPPT design system also includes a procedure for estimating the life cycle costs of alternative pavements.

The focal point of research in the TPPT project was on the pavement solutions and design of new high-volume roads. However, during the course of the research programme the emphasis of road management changed significantly from new road construction to maintenance of existing roads. As a result, a separate project concentrating on thin pavement roads was started alongside the TPPT project. At the same time the TPPT project was reduced down slightly.

Several projects with parallel, but more limited goals than the TPPT project were implemented alongside the TPPT project:

- Sustainable development structure idea competition
- Settlement calculation competition
- REFLEX steel mesh project
- Use of industrial by-products and development of life cycle assessment
- Instrumented test road in Temmes
- Thin pavement road project
- Damages of road structure and the condition of the road network

When the focal point of road management changed, the relative significance of these projects increased toward the end of the Road Structures Research Programme.

This publication is the summarising report of the Road Structures Research Programme. The content of the programme and main results from some projects have been described very shortly in different chapters. This report has been published also in Finnish.

Most final reports of the project can be opened from Internet on pages www.finnra.fi/tppt.

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1 MAIN CONTENT OF THE ROAD STRUCTURES RESEARCH PROGRAMME

1.1 Start-up and main phases of the research programme

The Road Structure Research Programme (S4) was started in 1994. The main part of the program consisted of the Road Structures Research Project (TPPT project) developing pavement and the substructures. The TPPT project was preceded in the 1980s by the ASTO project, which developed asphalt structures to increase their resistance to wear caused by studded tires. The ASTO project was very successful. Wear resistance of pavements was significantly improved, reducing the need for repaving.

Encouraged by the ASTO project, the same model was used to plan the extensive eight year TPPT project (1994-2001), which was implemented by the Technical Research Centre of Finland (VTT). In addition, road structures were developed and studied in several smaller separate projects having parallel goals with TPPT. The goal of the TPPT project was to improve the trafficability and durability of both new and restored roads in order to lower the annual cost of road management and minimize environmental impacts.

In 1995 VTT named a team of foreign experts to evaluate the TPPT project comprised of the following persons: Dr. Jan Hartlen (Swedish Geotechnical Institute / Lunds University, Sweden), Dr. Ronald W. Hudson (University of Texas at Austin, USA), Dr. Dave Newcomb (University of Minnesota, USA), Dr. Johann Litzka (TU Wien, Austria) and Dr. Rolf Magnusson (Swedish National Road Administration, Sweden). During the first years of the program, the opinions and expertise of these experts influenced the weighting of the various topics and also the details of the contents of certain topics. The team was active until 1999.

The focal point of research in the TPPT project was on the structural solutions and design of new high-volume roads. However, during the course of the research program the emphasis of road management changed significantly from new road construction to maintenance of existing roads. As a result, a separate project concentrating on thinly-pavement roads was started alongside the TPPT project. At the same time the TPPT project was reduced down slightly.

The main phases of the research programme were:

1992–1993	TPPT project planning
1994	TPPT project start-up
1994-1999	Laboratory studies; unbound, bitumen, composite materials
1995–1999	Construction of test structures at 19 sites; frost-resistant, composite structures, fatigue resistant structures

1995	Idea competition for a sustainable development road structure. Development work begins to pelletize till
1997–1999	Settlement calculation competition
1997	HVS-NORDIC tests started in cooperation with Sweden
1999-2001	Start-up of the "thin-pavement road project"
2001	Collecting of results; TPPT design system, descriptions of investigation methods, design guidelines

1.2 TPPT's content and results

The objective of the results obtained from the Road Structures Research Project is to be able to construct new roads and repair old paved roads more durable while reducing the annual cost of the structures. The goal of the projects was to develop the design methods of the pavements to make them more durable and to make it possible to estimate their performance during their entire life span. If the performance of the road structures can be estimated in advance, the risk of unexpected damages decreases.

The results of the TPPT project were compiled into a design system. The basic idea is presented in the form of a flow chart in figure 1.1. The design guidelines and descriptions of the design system present the procedures and methods with which a pavement and road foundations for a particular site can be designed to keep settlement, frost damage and load-bearing capacity under control. The TPPT design system also includes a procedure for estimating the life cycle costs of alternative pavements.

New structures were developed at test construction sites related to TPPT and with an heavy vehicle simulator (HVS Nordic, see chapter 7). The purpose of the TPPT test structures was to conduct actual full-scale tests of structures developed in the research program. Nineteen test sites were constructed in 1995-99, where different types of pavements and materials were tested on roads and pedestrian and bicycle ways. Test construction was used to study

- load-bearing capacity and frost resistance, changes in evenness and damage to structures under actual loading and climatic stress,
- behavior of materials under actual loading and climatic stress, and
- usefulness of developed design procedures and behavior models.

Observations and measurements made during the design, construction and monitoring of the test structures were also used to verify the developed design system and descriptions for pavements and soils investigations.

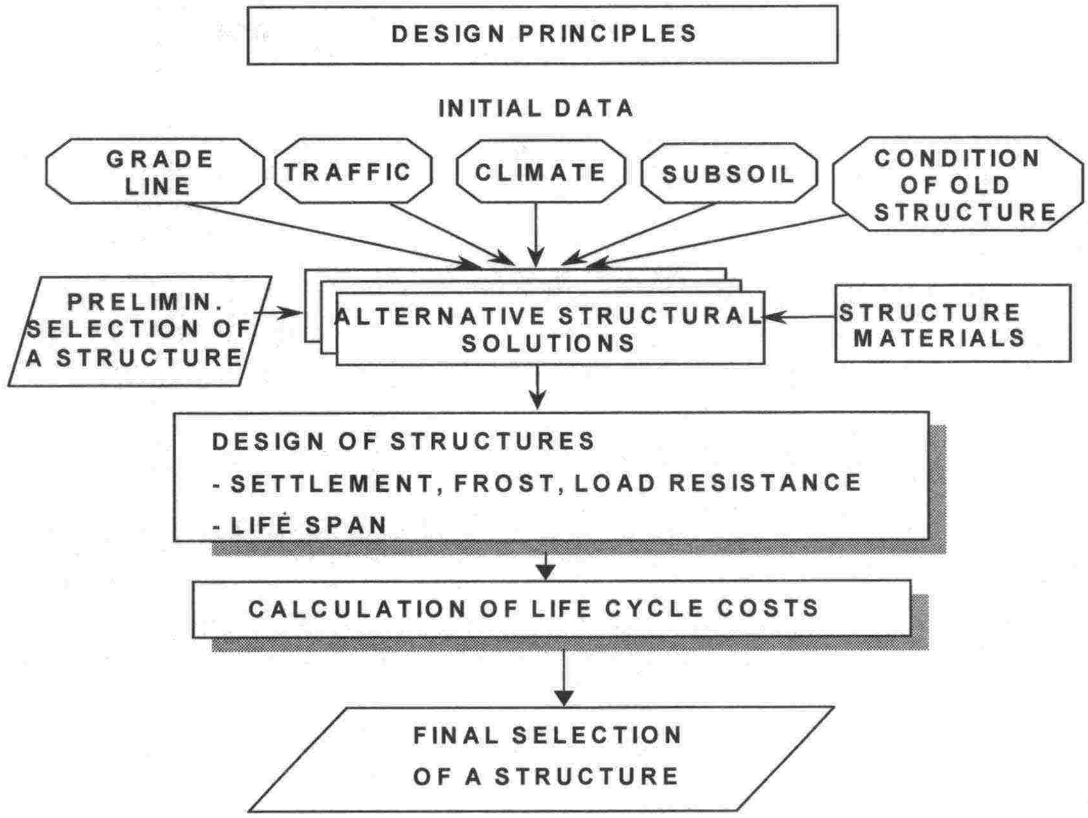


Figure 1.1. Flow chart of the TPPT design system.

Sod peat structures, a granulated blast furnace sand structure and an LD steel slag structure were studied at frost protection sites. Steel mesh structures, geoprofile (made of steel) structures, geosynthetic reinforcement nets and steel nets in soil cement structures were studied at sites with reinforced structures. Substructures were homogenized and stabilized using cement stabilization and Finnstabi™ + lime stabilization. The pavement and embankment of the old road were used in refining till in mixed till structures. Bitumen stabilization, composite structures and a Gilsonite structure were studied in load-bearing capacity pavements. The use of fly ash, a product of coal burning, was studied in the base course of one site.

Projects included in the road districts' yearly programs for 1995-1999 were selected as sites, which took into consideration reasons behind the need for improvement and different climate areas and road classes. The test construction program implemented and studied structural solutions for new and improved roads. Test structures were made in conjunction with the districts' normal construction, so the test sites always included a conventional structure as a reference.

Separate final reports of each test construction sites covering all the monitored years were compiled. The reports contain an estimate of the long-term performance and usability of the structures. A final summary report was compiled of all the test construction projects and their results / Kivikoski, H., Pihlajamäki, J. & Tammirinne, M. TPPT test construction sites.

Results. TIEH 8/2002 (in finnish) /. The report also includes proposals for follow-up monitoring of the sites after the TPPT project.

Since 1994 the TPPT project also included measurements and long-term monitoring on observation roads, of which most belonged to the SHRP–LTTP study. The measurements of the observation roads were used to develop deterioration models of pavement and assess the usability of the created models. Eighteen AC + base structures (asphalt concrete pavement on an unbound base) and 28 AC2 + base structures (repaved asphalt concrete pavement on an unbound base) were selected for this purpose. The current number of monitored sites is 41 / Spoof, H. Observation roads. TPPT Site report 43. 2002 (in finnish) /

Structural data (wearing surface thickness, structural layer thickness) and material data (material samples from each layer and the subsoil) were collected from the observation roads. Also, falling weight measurements (once or twice), damage inventories (type, quantity, seriousness) and PTM measurements (evenness and rut formation data) have been made at the sites every year since 1991. The material modules of the structural layers (wearing course, unbound layers and subsoil) were back-calculated using a Modulus-program, after which deformation and strains were calculated using a BISAR multi-layer program.

TPPT material studies focused on a very limited number of materials. The entire "product development chain" from mix design to material testing in test structures was implemented for only certain composite materials. Bitumen stabilized materials and crushed till with a cement binder were also studied in laboratory tests. Further studies on the use of till and development of materials for production will be done by Finnra. The only unbound structural layer materials studied in the TPPT project were crushed aggregates. The results of TPPT material studies from the standpoint of bearing capacity design are covered in separate TPPT reports /Materials in the structural layers of a road. Background material for material selection. TIEH 66/2001 (in finnish)/ and /Unbound materials in the structural layers of a road. Background information about material behavior. Report TPPT 22. 2001 (in finnish)/.

<u>TPPT design system for pavements</u>

In the TPPT design system a road being designed is divided into homogenous sections in terms of settlement and frost heave, and in the case of a road site being improved, also in terms of the load-bearing capacity of the old road. Settlement, frost heave and bearing capacity are taken into consideration in the design of the pavements. Calculation of settlement profile, frost design and bearing capacity design are presented as descriptions in the TPPT design system.

Settlement calculation is used to determine places along the road line where settlement exceeds the settlement criteria (total settlement, differential settlements, angular distortion). A method of calculating the settlement profile in the longitudinal direction of a road based on a TSARPIX program developed in the TPPT project can be used in designing new roads and improving old road in all road classes.

Frost design of pavement and frost insulation is based on the thermal conductivity of materials, subsoil frost susceptibility, the freezing index and frost heave criteria. Life cycle assessment based on frost heave risk (damage risk) can be used as a tool in selecting structural alternatives with different kinds of frost protection. The presented frost design can be applied in designing new roads and improving old road in all road classes.

Load-bearing capacity design of the pavement is intended for asphalt pavement (AC, ACK, SMA pavements) having no bound layers underneath the pavement. The TPPT design procedure assumes that the thickness of the asphalt is at least 60...80 mm, in which case asphalt fatigue is a factor that leads to damage. The bearing capacity design of the pavement is based on the horizontal tensile strain or deflection difference (SCI300) of the lower surface of the pavement, which explains how pavement fatigue leads to traffic-induced damage.

The performance of the pavement can also be examined by taking into consideration the bearing capacity of the subsoil during thawing, when the subsoil module is at its lowest value. The risk of rut formation in the subsoil can be estimated on the basis of the total deformation of the subsoil.

The life cycle cost estimation procedure for pavement, which is a part of the TPPT design system, is used to calculate the life cycle costs of designed alternative pavements. These comparisons of economy support decision-making when selecting a structural solution for a site.

Design criteria and TPPT limit values applied in designing are specified on the basis of road evenness (road user experience of driving smoothness) and the structural durability of the road and the pavement in particular. The limit values that are used should be specified so they can be calculated or estimated in advance and unambiguously measured from the road.

It is essential to the design procedure of the TPPT design system that designing is based on site-specific data and parameters (traffic, climate, subsoil, structural materials used). To unify initial data acquisition, TPPT descriptions were compiled. They were published as separate reports and are referred to in the guidelines. Observations and tests showed the recommended procedures to be useful. The design procedures also refer to various commonly used guidelines published by Finnra and other organizations, which are not dealt with in conjunction with TPPT. The list of giudelines and descriptions, as well as other TPPT-publications is in Appendix 1.

1.3 Other projects

Sustainable development structure idea competition

In 1995 a public idea competition was held to get new ideas in road construction. Over 100 new or unestablished proposals were received for pavements or superstructures. The winning idea proposed refinement of till by pelletizing it with cement or bitumen. The idea was developed further by means of laboratory studies and field tests. The results of the development work are described in chapter 5.10.

Settlement calculation competition

The accuracy of settlement calculations of road embankments constructed on soft ground was studied by arranging a calculation competition in 1997 - 1999. Several Finnish and foreign participants predicted the settlement of the Haarajoki test embankment using conventional and modern methods. The deviation of the results was considerable. More about the competion is in chapter 7.1.

REFLEX steel mesh project

Steel mesh has been successfully used over thirty years in Finland to correct longitudinal frost-induced cracking of roads. A European research project studied the possibility of also using steel mesh to improve the load-bearing capacity of a road. Finnish researchers and Finnra also participated in the project. The preliminary results are promising. The results of the study will be published in 2002.

Use of industrial by-products and development of life cycle assessment

Coinciding with the TPPT project, The National Technology Agency of Finland (TEKES) financed an "environmental geotechnology program". It developed environmentally sound earth construction and use of by-products. Finnra provided funds for experimental construction using by-products and development of life cycle assessment (LCA). Development of by-products is presented in chapter 7.2 and life cycle assessment in chapter 2.4.

Instrumented test road in Temmes

An ordinary pavement in Temmes, near Oulu, was equipped with wide-ranging instrumentation in 1996 – 1997. The test road in Temmes has been used to study the impact of traffic and climate factors on pavement stresses.

The results were used, e.g., to create a calculation model for specifying the average temperature of the pavement.

Thin-pavement road project

The most significant parallel project was a three-year thin-pavement road project. It developed new design procedures and condition assessment procedures for roads with a thin pavement (< 80 mm), which differ from roads with a thick pavement. The results of the projects are presented in chapters 2 and 5.6.

Damages of pavement and the condition of the road network

The complexity of the damage process of a pavement has hindered assessment of the development of a road's condition as well as discussion about the issue. The study described the behavior of the road structure and factors that affect it simply and illustratively. The study also analyzed the condition of the road network. The subject is covered in chapters 8.1 and 8.2.

MnRoad cooperation

MnRoad, an experimental road equipped with wide-ranging instrumentation, was constructed in Minnesota (USA) in 1990-91. It consists of part of a motorway with 83 different structures and a separate low volume road with 17 separate structures. Because the geology and climate of Minnesota are quite similar to those of Finland, experiences obtained from MnRoad were utilized in the TPPT project. Correspondingly, some Finnish solutions have been tested in Minnesota. Cooperation between the countries has also included researcher and trainee exchanges and visits by experts. Information about MnRoad and its results are available at:

`http://www.dot.state.mn.us/` or
`http://mnroad.dot.state.mn.us/research/MnROAD_Project/MnROADProject.asp`

2 PROJECT LEVEL LIFE CYCLE MANAGEMENT

Road life cycle management is divided into life cycle cost calculation (technical-economic effects) and life cycle assessment (environmental impact). Life cycle cost calculation is used to calculate the road owner and road users' life cycle costs for all the pavement alternatives being examined and thereby select the solution with the lowest annual costs. Life cycle assessment consists of assessing a road pavement's environmental impact from material production and construction of the pavement to the effects of its use and dismantling and recycling.

2.1 Calculation of life cycle costs

In planning the structural designs of a road, investment costs have traditionally steered the selection of the structures used, meaning that the least expensive alternative from the standpoint of investment costs has been chosen. The TPPT project has compiled a calculation system for AC and PAC pavements that takes into consideration the investment costs, maintenance costs and road user costs over the entire pavement life cycle. In calculating life cycle costs it is necessary to be able to estimate the life span of the structural design being examined. For this reason, performance models have been compiled for ordinary AC and PAC pavements. The performance models for other pavements, such as bitumen and cement stabilization and new structural designs are incomplete.

Because the rehabilitation costs of the foundation are considerably higher than the costs of the pavement, it is recommendable to calculate the life cycle costs separately. In principle, calculation of life cycle costs is similar for new constructions as for rehabilitation of existing pavements.

By calculating life cycle costs it is possible to optimize:

- cost-effectiveness of alternative pavement designs
- cost-effectiveness of two different maintenance strategies

In optimizing the economy of alternative pavement designs, the annual costs of different pavement alternatives over the entire life cycle are compared. By comparing strategies it will be searched search for an optimal maintenance level, for example, the level to which the condition of a road is allowed to decrease. For example, an analysis can compare whether it's more cost-effective to perform less expensive, light maintenance measures frequently or less frequent, but more expensive, heavy maintenance measures (figure 2.1).

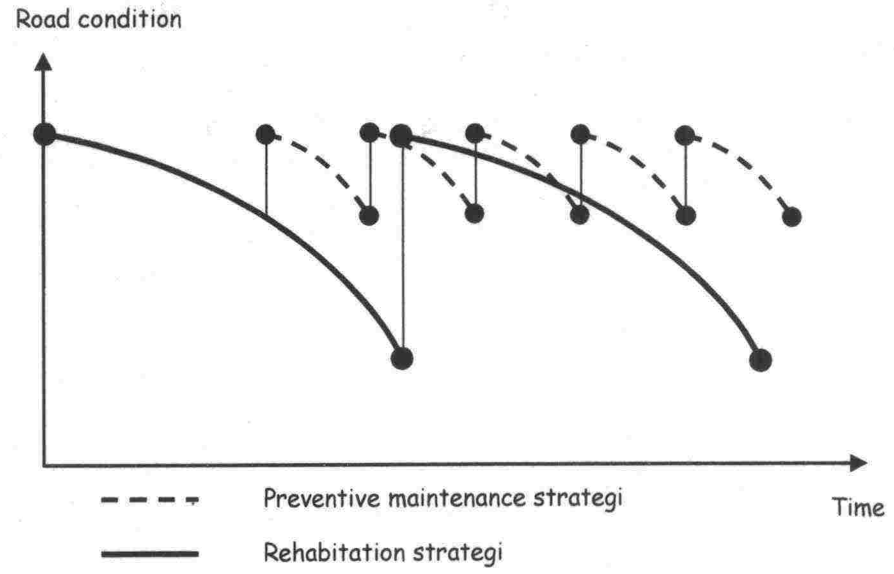


Figure 2.1. Two different road condition maintenance strategies.

<u>*What is included in the calculation ?*</u>

Life cycle cost calculation takes into consideration the following road owner and road user costs. Cost items with no difference between different structural designs can be left out of the actual calculations:

Road owner costs:

- Construction or rehabilitation costs, i.e., investment costs including planning costs
- Costs to the road owner of maintenance measures
- Annual routine maintenance costs

Road user costs:

- Costs to road users of future maintenance treatments
- Annual user costs

Estimated costs for each year are discounted to their present value and added together. The pavements are compared on the basis of the annual cost over the period being examined.

<u>*Length of the calculation period*</u>

The life cycle of different parts of a road may be very different. The pavement of highly trafficed roads in Finland may need to be maintained with few years intervals because of rutting due to wear from studded tyres. The target life span of the foundation may be 50 – 100 years. Different rehabilitation and maintenance treatments have different life expectancy. In this context the life cycle of a road refers to the length of the period being examined, which the designer selects for a cost comparison of alternative structural designs and/or maintenance treatments done to a road. The period being examined may be 20, 30 or 40 years, for example. Because the life expectancy of the PAC pavement is usually shorter, the examined period for roads with thin layer is shorter than for roads with thick layers.

The longer a period of examination is selected, the less accurate the estimation of the cost of future maintenance treatments becomes. The farther in the future an estimated maintenance treatment is, the smaller is its impact on total costs, because costs are converted to their present value with the help of a discounting factor. The higher the selected interest rate, the smaller is the impact.

Life cycle cost calculation system

The initial data for the TPPT life cycle cost calculation system shown in figure 2.2 are obtained as design criteria, intervention levels and design results of the selected strategy.

The investment costs of a designed pavement solution are taken into consideration in item 1 of the calculation flow chart. Then the budget limit is checked before moving downward in the flow chart. If the budget framework is exceeded, the design must be changed to fit within the allowed limits.

After this the loop in the middle of the flow chart is followed in the clockwise direction by making sure the threshold levels specified by the strategy are not exceeded and by cumulating the annual routine maintenance costs and user costs of items 2 and 3. After this, if the period of examination has not ended, one year is added, traffic growth is checked and new condition parameters are predicted. This loop is followed until one of the threshold levels specified in the strategy is exceeded. Whereupon the loop bypassed down the center and the maintenance costs are cumulated in item 4, the annual routine maintenance costs are cumulated in item 5 and the annual user costs and additional costs of roadworks are cumulated in item 6. When the loop has been followed until the period of examination has ended, the annual cost for the life cycle of the pavement is calculated.

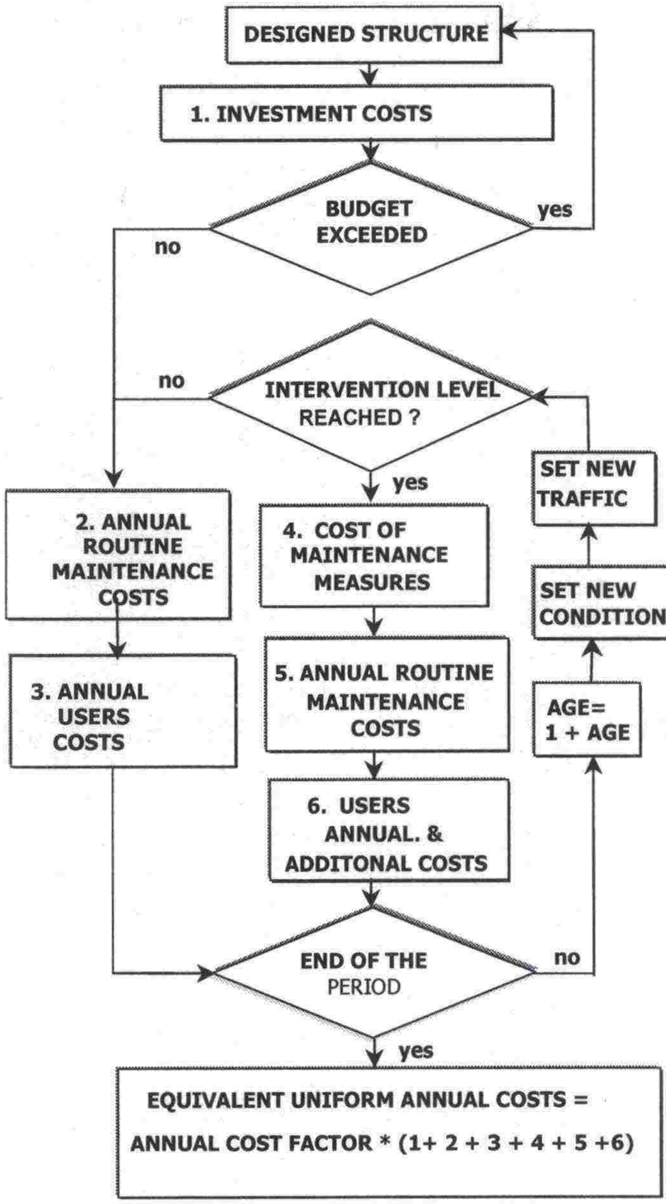


Figure 2.2. Life cycle cost calculation flow chart (TPPT pavement design system).

Threshold levels

Four performance indicators are used in road maintenance management and scheduling:

- distress (distress index)
- rutting (rut depth)
- longitudinal unevenness (IRI)
- bearing capacity.

Each performance indicator has certain threshold levels, which depend on road class, traffic volume, etc. Bearing capacity does not have threshold level, because the degree of bearing capacity has been found as a poor indicator to describe of the structural condition of the road or to predict the propagation of pavement distress. Table 2.1 contains examples of threshold values of maintenance measures /7/.

Table 2.1. Threshold values of surface performance indicators (PMSPro basic settings /7/).

	AADT (vehicles/day)			
	>6000	6000-1500	1500-350	<350
Rut depth, mm	15	16	17	18
if speed limit < 80 km/h	16	17	18	19
Unevenness, IRI mm/m	2.5	2.5	3.5	3.5
if speed limit < 80 km/h	2.5	3.5	4.1	4.1
Distress index, m^2 AC	30	50	70	140
PAC	40	60	80	140

The development of performance indicators is predicted with performance models based on field observations. In calculating life cycle costs it is necessary to estimate the effect of maintenance treatments on each condition variable separately. Rutting caused by pavement wear and deformation is the most common type of deterioration on road with high traffic levels. Prediction of condition is discussed later under "Performance prediction models".

Calculation of costs during the analysis period will be made as follows (grouped according to the calculation flow chart). In calculating the costs of the analysis period only the factors in which the alternatives differs from each other are taken into consideration.

2. Annual routine maintenance costs: Usually not taken into consideration.

3. Annual user costs: Normal costs during the analysis period, such as travel time costs, vehicle operating costs and accident costs. These are usually not taken into consideration. Differences in condition on the low volume road network may affect travel time and vehicle costs.

4. Costs of maintenance measures are calculated according to planned procedures.

6. Additional costs to users caused by maintenance measures are usually taken into consideration. Maintenance measures causes additional user costs due to congestion and extra travel time when passing a roadwork area or higher vehicle costs and longer travel times due to detours. When traffic volumes are high, the additional user costs caused by a worksite may be a reason to avoid disturbance of traffic by using designs that with longer life expectancy.

Cost calculation is presented in more detail in the TPPT procedure description TPPT-20 "Life cycle analysis of pavements" /1/

Pavement residual value

The residual value of a pavement is the present value of the pavement that depends on its condition. It can also be treated as a cost that is necessary to restore the pavement to its original condition. If the analysis period of the life cycle cost calculation is very long (over 30 years), the difference in the residual values of alternative pavements are not significant anymore for the results.

2.2 Performance prediction models

Performance models are used to assess the situation when a pavement reaches its threshold level (intervention level). Predictable performance indicators are:

- pavement distress
- rutting (transverse profile)
- longitudinal unevenness (IRI).

Development of these performance indicators is predicted with different methods for pavements with thick asphalt layers and roads with thin asphalt layers.

Practical application of the life cycle cost calculation method is hindered by the lack of initial data for some partial factors and deterioration models are still incomplete. The effect of maintenance treatments on the future performance of the road is not known accurately enough yet.

Thick (> 60...80 mm) AC pavements

Pavement distress

Modeling of pavement distress consists of three phases, figure 2.3.

- distress initiation caused by traffic loading after construction and climatic distress during that same period of time (design period).
- distress propagation caused by climate and traffic (after distress initiation)
- distress propagation caused by climate and traffic after application of maintenance measure

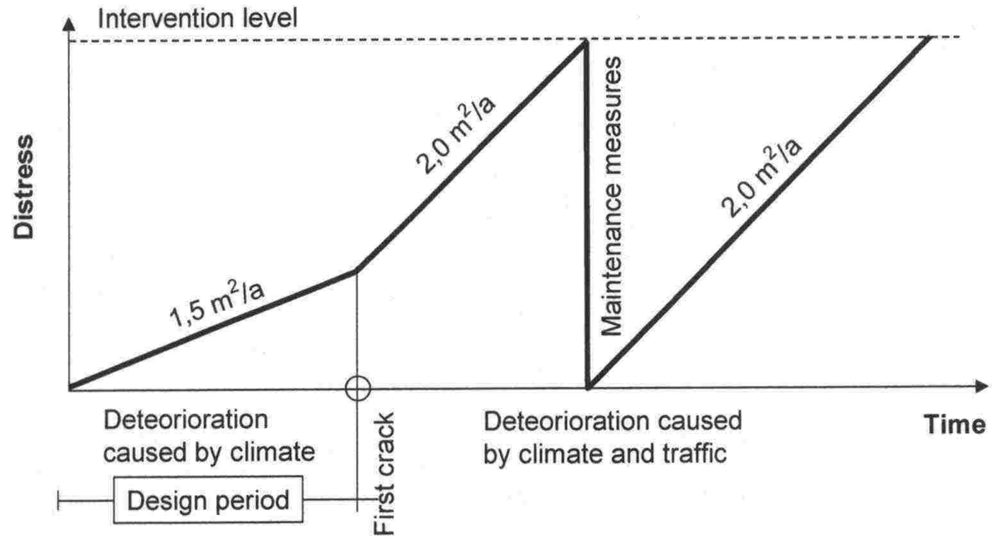


Figure 2.3. Distress propagation during life cycle.

- The design criterion for pavement distress is the initiation of cracking in the wheel path. In the model used to calculate the time when crack initiation appears (see chapter 5), the variables are the horizontal strain in the bottom of the asphalt layer and the annual average daily traffic (AADT). The crack initiation model reliably predicts the time when distress caused by traffic loading starts to appear in the wheel paths of a road with thick asphalt layers, but the model still has some limitations.

After a road is constructed it is subjected to climatic stresses (mainly frost), which deteriorates the road. The rate of deterioration caused by climate is most affected by the thickness of the pavement layers, the frost heave characteristics of the subgrade and the climate zone. If the designed frost heave is 50 mm, the highest rate of deterioration independent of the subgrade and climate conditions is in TPPT system a constant of 1.5 m^2/a. If the designed frost heave is greater than 50 mm, the deterioration rate caused by climate need to be calculated.

<u>Distress caused by climate and traffic after distress initiation</u>

The same deterioration rate is used in the calculation as is used after maintenance is applied. It includes distress caused by both climate and traffic (table 2.2).

Deterioration rate after application of maintenance measure

The Finnish Road Administration has studied how different structural factors and climate and traffic-based factors affect the rate of deterioration after application of maintenance. No structural factor or load factor in the studied material explained the rate of deterioration after maintenance. The rate of deterioration was 25 % higher on sites where a thin surface was applied than on sites where the maintenance measure was repaving. Average deterioration rates for AC roads with thick asphalt layers are presented by traffic volume class in table 2.2.

Table 2.2. Rate of deterioration by traffic volume class after maintenance measure.

Measure	ADT > 6000	ADT 3000 - 6000	ADT 1500 – 3000
Thin surfacing	1.9 m^2/a	2.2 m^2/a	2.5 m^2/a
Repaving	1.5 m^2/a	1.8 m^2/a	2.0 m^2/a

This model does not explain the effect of the present condition on future deterioration. The prediction of distress propagation after application of maintenance measures is still at a rough level

<u>Pavement rutting</u>

The method of calculating the overall rate of rutting of asphalt layers is based on the results of the ASTO experiment made during 1990 – 1997. Rutting can be divided into two parts: asphalt wear caused by the studded tires of passenger cars and permanent deformation caused by heavy vehicles. An Excel application, PCAD, has been created for calculation of rutting. Asphalt rutting models are relatively accurate.

Initial data needed for the wear model:

- Number of passenger cars and heavy vehicles
- Road width, speed limit
- Climate zone
- Mixture type
- Ball mill value of aggregate
- Binder type

Initial data needed for the deformation model:

- Number of heavy vehicles
- Mixture type
- Binder type

<u>Longitudinal unevenness</u>

TPPT's life cycle calculation is based on models used in road maintenance management and programming systems (HIPS/PMS) to model the performance of longitudinal unevenness by traffic volume class. As a result of an analysis of PMS's deterioration models, the following longitudinal unevenness model is recommended for asphalt roads (2.1):

$$dIRI = 0{,}016 + 0{,}0524 * IRI(t), \qquad (2.1)$$

where,

dIRI is the annual change in longitudinal unevenness
IRI(t) IRI in the year t

Prediction of changes in longitudinal unevenness is quite reliable.

Thin (< 80 mm) PAC pavements

Transverse unevenness

On paved narrow roads the height of the ridge between ruts has proved to be a better indicator of transverse unevenness than the depth of the outer rut. Especially on narrow roads, the widening and rutting mechanisms do not noticeably slow down with age, but rather the height of the ridge grows linearly every year. During the thinly pavement road project, separate models were developed for the rate of ridge growth on PAC pavements in southern Finland (F_{10} < 40000 Kh) and northern Finland (F_{10} > 40000 Kh) and for roads with thin AC pavement in southern Finland /2/. The variables used in the models are pavement width, deflection difference d0 – d450 and average daily traffic (see chapter 5.6). Testing of the models is not complete yet.

There are no threshold levels for ridge height because it hasn't been used yet as a performance indicator in the Finnish Road Administration's management system. In TPPT life cycle calculation 25 mm has been used as a intervention level for structural rehabilitation.

Pavement distress

In the project dealing with the development of pavement performance models for pavements with thin asphalt layers, the same variables were used to model the distress index as were used to model propagation of ridge height (see chapter 5.6). A distress index prediction model was developed for use in road maintenance management systems when the present condition is known.

The method described above for the annual propagation of the distress index on roads with thick AC layers can also be applied to roads with thin pavements /2/.

Longitudinal unevenness

As a result of an analysis of PMS's deterioration models, the following longitudinal unevenness model is recommended for roads with thin pavements (2.2):

$$dIRI = 0{,}036 + 0{,}0560 * IRI(t), \qquad (2.2)$$

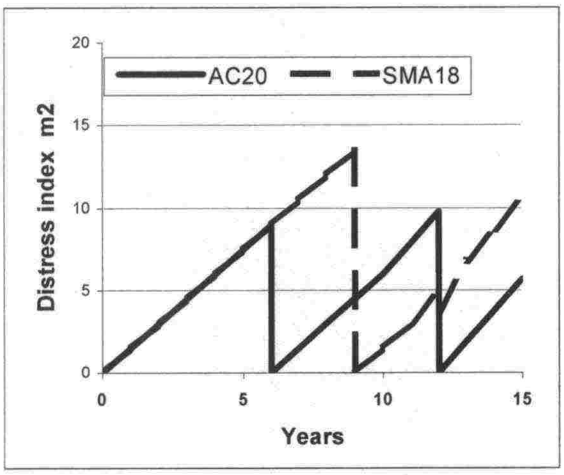
where

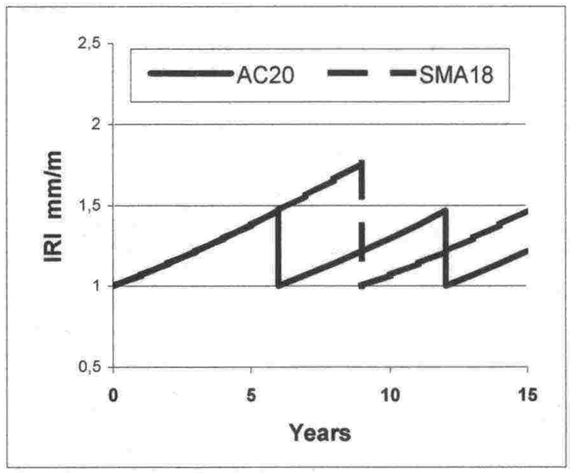
dIRI is the annual change in longitudinal unevenness
IRI(t) IRI in the year t

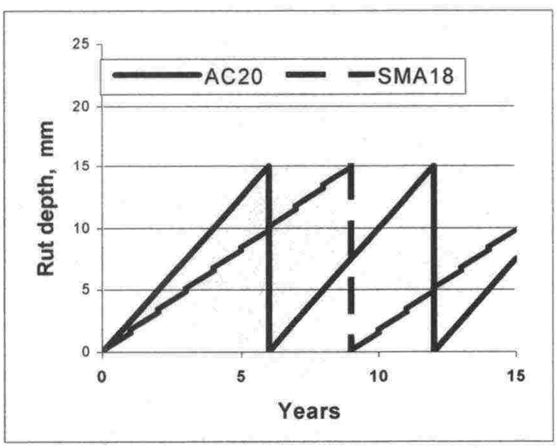
The initial IRI value is 1.0 mm/m for roads with thick AC layers, 1.3 mm/m for thin AC layers and 1.5 mm/m for PAC pavements.

Example

The effect of the type of pavement and the strength of the aggregate in the surface layer on pavement wear and thereby on annual costs is examined in the following example /1/. The first alternative has an AC20 surface layer with strength class II aggregate. The second alternative has a SMA surface layer with a strength class I aggregate, which is transported 100 km further than the aggregate used in the AC pavement. The changes in rutting, longitudinal unevenness and distress index over a period of 15 years are shown in figure 2.4. In both cases the trigger for maintenance is rutting due to wear of the surface layer.







Layer	Thickness (mm)	Module (MPa)
AC/SMA + ACB	200	4000
Load-bearing, crushed gravel	200	240
Separating gravel	300	160
Filter sand	200	70
Subsoil: sandy till	-	100

Maintenance class
ADT > 6000 vehicles/day intervention levels:
Distress index, VS > 30 m^2
Rut depth > 15 mm
Unevenness, IRI > 2,5 mm/m

Figure 2.4. The initial pavement data and the propagation of rutting, longitudinal unevenness and distress on an example site.

Table 2.3 presents the construction costs of the AC pavement, the estimated cost of maintenance and the additional road user time and vehicle operating costs (overall costs) over the analysis period of 36 years. The additional maintenance costs and additional road users costs are discounted to their present value. A 4 % interest rate is used in the calculation.

Table 2.3. Example of the results of a life cycle cost study (AC pavement).

Year of procedure	Type of procedure / cost	Cost euro	Discount factor	Discounted cost euro
0	Construction	328,600	1	328,600
6	Remixer Additional user cost	23,570 2,030	0.790	18,630 1,600
12	Remixer Additional user cost	23,570 2,030	0.625	14,720 1,270
18	Repaving Additional user cost	54,670 2,030	0.494	26,980 1,000
24	Remixer Additional user cost	54,670 2,030	0.390	9,180 800
30	Remixer Additional user cost	54,670 2,030	0.308	7,270 630
Total costs of analysis period discounted to present value				**410,680**

The type of maintenance measures is selected on the basis of annual costs. The use of annual costs makes it possible to use a different analysis periods for different types of pavements. The most cost-effective pavement has the lowest annual costs. The residual value near the end of the long analysis period is close to zero and it is not taken into consideration in these calculations.

The annual cost of the AC pavement is:

$V = c_n * K = 0.0529 * 410{,}680$ euro = 21,725 euro,

where

c_n	=	annual cost coefficient
K	=	present value of all costs

The annual cost of the SMA pavement is 21,550 euro. This is about one percent less than the annual cost of the AC pavement.

The present values of the overall costs are divided into investment, maintenance and additional user costs. In the AC pavement they are distributed as follows:

Investment costs	328,600 euro	80 %
Maintenance costs	6,780 euro	19 %
Additional time and vehicle operating costs	5,300 euro	1 %

2.3 Life cycle assessment as a part of road design

The criteria used in selecting products and materials are increasingly related to environmental impact during the life cycle. Life cycle thinking is well suited to road and earth construction. It can be used to develop structures and construction techniques that are environmentally more sound and to select earth construction materials that are environmentally more economical.

In life cycle assessment, material flow and emissions are specified for all phases of the life cycle of a product or procedure. The most significant hazards and the factors that influence them are identified.

The main phases of a life cycle that are assessed are:

- procurement of raw material
- manufacture of a product or material
- transportation and distribution
- use and maintenance
- reuse
- waste treatment and disposal.

The basic phases of assessment are specification of goals, inventory or calculation of material flow and emissions, estimation of impact and if necessary, estimation of possibilities of improvement. Because in all phases it is possible to make choices and assumptions that affect the results, it is important to report initial assumptions and results in such a way that the information can be checked. Therefore, the analysis should also include sensitivity and uncertainty analyses, which are used to check how much the results depend on the choices that are made.

The objective is to plan a road project or individual structure that loads the environment as little as possible, while taking into consideration its environmental load during its entire life cycle from raw material procurement to possible removal from use.

Life cycle assessment can be utilized, for example:

- as a decision-making aid when comparing different structures or materials.
- in assessing the need for improvement and in selecting the most cost-effective and environmentally efficient procedures when developing own production.
- in product marketing, where customers can be given life cycle information about products they use in their own operation or comparative information about the environmental load produced by different alternatives.

- in applying for environmental permits and in cooperation with authorities, e.g., to indicate the overall environmental impact of different alternatives and to show the benefits and advantages of different alternatives.

2.4 Life cycle assesment program, MELI

In TEKES's environmental geotechnology project it has been compiled an Excel-based life cycle calculation program for road pavement structures and substructures, MELI /4,5,6/. Being easy to use, the calculation program is well suited for use in designing road structures.

The study first compiled a procedural model for assessing the environmental impact and comparing the structural alternatives during the life cycle of road and earth structures. The procedural model was applied in example structures to assess environmental loading. The use of coal ash, crushed demolition concrete, blast-furnace slag and natural aggregates in road construction was compared using example structures with similar technical properties.

Environmental load data of the most important construction materials and work phases were gathered in conjunction with the examples. The data were combined into an application that facilitates later calculations and comparisons. As initial data the user enters the dimensions of the structure, the materials used and the transport distance of the materials. For estimation of maintenance measures, the type of maintenance and the number of maintenance measures are also entered. With the calculation program it is possible to produce estimates of the environmental loading of the entire structure or certain construction phases, either as is or according to a point system.

Figures 2.5 and 2.6 present comparisons of material and energy consumption and carbon dioxide and nitrogen oxide emissions in road structures planned for the same one-kilometer-long road structure. In figure 2.6 the energy consumption is also presented divided into main work phases: energy consumption during material production, transport and road construction.

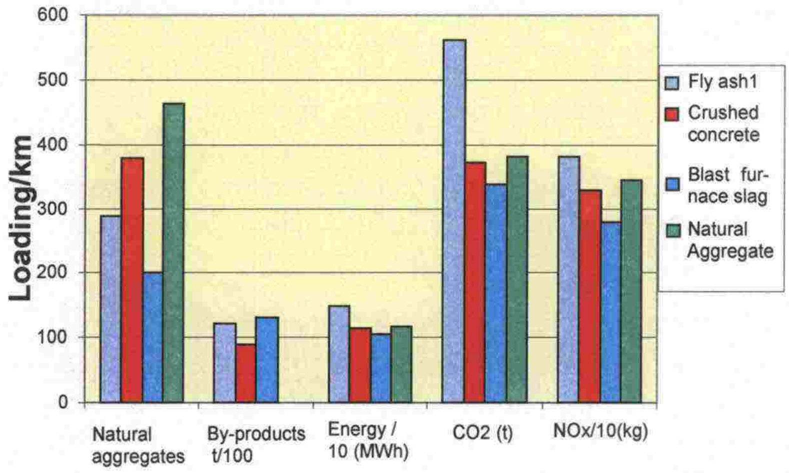


Figure 2.5. Use of material and energy and carbon dioxide and nitrogen emissions in alternative road structures

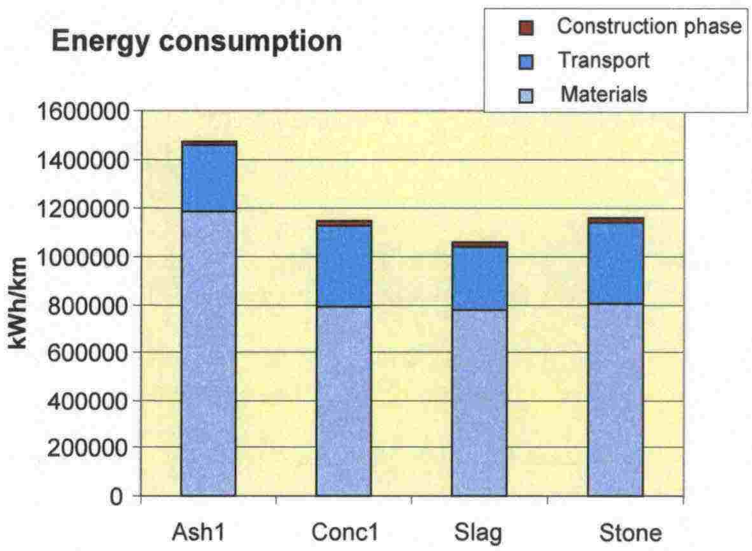


Figure 2.6. Total energy consumption in the structures of figure 5 divided by main work phase

The alternative structures of figures 2.5 and 2.6 are:

- Ash1: fly ash + 2 % concrete mixture in the separating layer
- Concr1: crushed concrete in the load-bearing and separating layer
- Slag: crushed blast-furnace slag and blast-furnace sand in the load-bearing and separating layer
- Stone: only natural aggregate.

2.5 Applying life cycle thinking in practice.

The Finnish Road Administration is developing its operation so that planning and construction strive for overall economy and minimization of undesired environmental impacts. The bidding procedure is being developed so that contractors compete with the life cycle characteristics of their products. Entrepreneurs are also expected to bring alternative, ecologically sustainable products to the market.

Life cycle cost calculation and life cycle assessment must be incorporated as a basis for making decisions involving planning and construction. The development work that has been done has created the prerequisites for making life cycle calculations and initiating pilot contracts that require life cycle know-how: They are good for encouraging other parties to develop their own life cycle know-how. The most important areas of development are life span models of road structures and life cycle calculation tools.

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3 CALCULATING A ROAD'S SETTLEMENT PROFILE

In the TPPT design system /2/, control of road embankment settlement is based on a continuous settlement profile formed from the expected settlement of the road embankment /4/. A calculated continuous settlement profile can be used to determine settlement areas on the road line requiring ground improvement, areas requiring transition structures and areas that do not require any measures, on the basis of criteria specified for total settlement and settlement differences.

To define the continuous settlement profile of a road embankment it is necessary to define and describe continuous ground conditions and load data along the road line and employ a settlement calculation program to process the data. The most significant change in the TPPT design system compared to the currently used settlement calculation procedure is the use of a continuous presentation of ground conditions and a continuous settlement profile based on the ground conditions. The common procedure is to calculate settlement and present the results as point information.

3.1 TSARPIX settlement calculation program

In the TPPT design system, the road settlement profile is calculated using a TSARPIX program /4/. The calculation procedure is based on the creation of a continuous settlement soil structure model depicted as a pixel model. The pixel model's initial data describing soil settlement properties is primarily obtained from water content tomography created by means of electrical resistivity sounding. The TSARPIX application is supplemented by a RAIPIX program (processing and correction of electrical resistivity sounding results) that processes the initial data and an Excel-based TSARPIX program used to create initial load data. Additionally, to process and visualize the data, an program like Surfer is needed to produce the pixel grid for the level curves and the calculation model.

The pixel model used to calculate the settlement profile differs from ordinary settlement calculation procedures in that the characteristics of the elements used in the pixel model, i.e., the pixels, differ from each other in the horizontal and vertical directions. On the level (longitudinal and cross directions of the road), the soil structure model is made up of small rectangular (cubical) elements. Most of the settlement characteristics of each element are defined with the help of water content tomography created using electrical resistivity sounding, and the rest are created using supplementary site investigations. In the TSARPIX program, settlement calculation does not, however, base on the water content method. Instead, the water content data of the ground are converted to parameters of a tangent modulus procedure. The principle (cross section) of a pixel-based soil structure model is presented in figure 3.1.

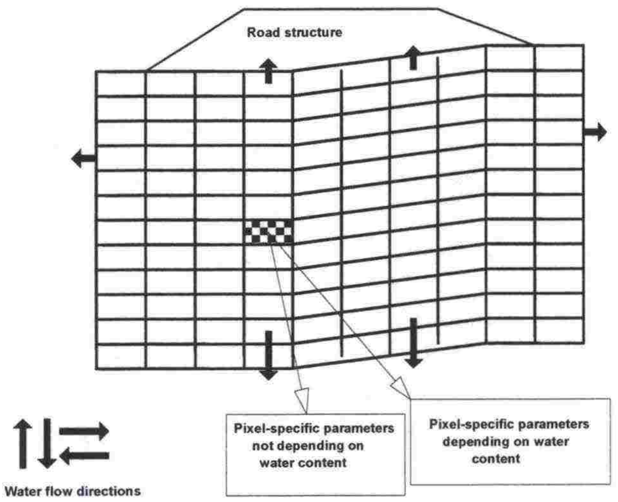

Figure 3.1. Principle of a pixel-based soil structure model, cross section.

3.1.1 Formation parameters of a pixel model

The specific resistances obtained through soil resistivity sounding are converted into water content values and the water content space is converted into settlement parameters.

The preconsolidation pressure σ_c and other settlement parameters that aren't dependent on water content are defined separately at the center of each pixel. The boundary conditions for the calculations are also defined manually. The boundary conditions are pore water overpressure and/or pore water flow rate. With the TSARPIX program it is also possible to take into consideration the soil layers that conduct water well.

Other information, such as load boundary conditions, needed in settlement calculation are created according to normal settlement calculation practice. A separate Excel-based program (TSARPIX) has been made to prepare this initial data for each mileage post. This program uses the water content data obtained by means electrical resistivity sounding to create corresponding mileage post-specific initial data on the level of the surface of the ground, the groundwater level, the level of the grade line, the excavation level (cut level; default ground surface) and the amount of any lightening at a given point in the road.

3.2 Ground resistivity sounding in obtaining initial data for settlement calculation

Electrical resistivity sounding is a non-destructive, geophysical procedure that is generally used to obtain information for a continuous description of the ground's characteristics /3/. The procedure is based on the electrical conductivity of material. The electrical conductivity of mineral soil is most significantly dependent on the quantity and salt content of pore water. Thus, the specific resistivity of clay is significantly smaller than that of coarse-grained soil layers. The accuracy to locate different soil layers depends on, among other things, the distance between electrodes when making measurements.

Unlike ground penetrating radar sounding, for example, with electrical resistivity sounding it is possible to obtain information from below soil layers with a high water content, making it possible to identify the location of non-compressible soil layers or the bedrock surface below clay if the differences in electrical conductivity of the layers are large enough. Any boundary layers, such as water conducting layers, inside compressible layers are identified with other soil investigation procedures. Other site investigation methods are also needed to obtain other information needed in settlement calculation and to specify site-specific settlement characteristics in more detail.

Electrical resistivity sounding as such, or interpreted as a water content value, can be used alongside normal investigation practice to select sounding and boring points or to change the distance between points, for example. Even without settlement calculation based on electrical resistivity sounding, continuous ground description has been found to be beneficial in practice.

3.2.1 Measurement and tentative processing of the results

Defining the specific resistivity resistance distribution is based on measuring the electrical resistance using Wenner's α-method. Electrodes are placed in line on the surface of the ground, usually equidistant from each other (e.g., VTT's equipment contains 52 electrodes). A computer feeds current into two electrode pairs at a time and measures their potential difference. By feeding current into electrodes close to each other and measuring their potential difference it's possible to obtain information about the surface layers of the ground. By feeding current into electrodes farther apart from each other and measuring their potential difference provides it's possible to obtain information from deeper in the ground. Due to the measuring procedure, one layout gives a wedge-shaped resistance description of the ground. The distance between the electrodes defines the length and depth of the layout and the resolution of the measurement. By performing several partly overlapping layouts it's possible to obtain the specific resistance distribution of the ground over the entire desired area.

With electrical resistance sounding it's possible to measure the electrical resistance of the ground at a certain depth below the surface, i.e., at the depth of the measurement electrodes. The results of the measurement must be converted to the actual elevation using topographic correction. The results of individual layouts also need to be combined with each other. In conjunction with these procedures it is also necessary to check and if needed, correct even any errors or deviations in the measurement results. This is done with the RAIPIX program.

Tentatively interpreted measurement results can be used to evaluate the need to expand electrical resistivity sounding. In that case interpretation of soil's settlement sensitivity is based on the resistance distribution obtained using electrical resistivity sounding /3, 4/. Roughly speaking,

- when the resistance R > 500 Ωm, the soil layer can be considered non-compressible
- when the resistance is 100 Ωm < R < 500 Ωm, the expected compression of the layer is moderate and
- when the resistance R < 100 Ωm, compression is significant.

3.2.2 Water content conversion

To convert the specific resistances to water content values, information is needed from the water content profiles of separate measurement points measured with electrical measurements. Because the specific resistance of a clay layer depends not only on water content, but also on other factors like the mineral content of the clay, the water content conversion is made at least for each separate clay basin. The number of measurement points is estimated on the basis of the sedimentation process. The tentative results of the electrical resistivity sounding are used to select the location of reference points for the site-specific water content conversion. The reference points are located in places where the resistance value is less than 500 Ωm and preferably where all the area's compressible soil layers exist. A measurement result should at least be obtained from the softest layers.

Water content is determined for each point using either radiometric gamma and neutron measurements or samples from continuous sampling. The specific resistivity is defined for the conversion from the result of an electrical resistivity sounding, a separate electrical point sounding or a specific resistance measurement of an undisturbed sample. A procedure based on radiometric measurement is more recommendable because it can be used to define the water content profile almost continuously and it also provides other necessary design information.

A site-dependent specific resistance – water content correlation is formed by defining a matching function for the resulting pair of points. With the RAI-PIX program the matching can be done by utilizing all the different water content values. By using the site-dependent specific resistance – water content correlation the continuous specific resistance correlation distribution can now be converted to a continuous water content distribution. Figure 3.2 contains a diagram showing how the electrical resistance distribution is converted to water content values (example).

3.2.3 Further processing of the measurered data

The RAIPIX program is used to form a suitable/desired calculation network for a pixel model /4/. As an example, in a case study presented later in this chapter the initial data for the settlement calculation are formed into 1 m ×

0.5 m (horizontal × vertical) pixels. Other pixel sizes are also possible. Their use may be justified in the depth direction especially with thin layers, and in the longitudinal direction, for example, to calculate values describing the evennes of the road. Also during the further processing phase initial data are processed, and generated values are weighted in the horizontal direction to take into consideration the layer-like structure of the ground. The Surfer-program was used to generate pixel-dependent specific resistance values for settlement calculation from the coarser pixel network of measured specific resistance values.

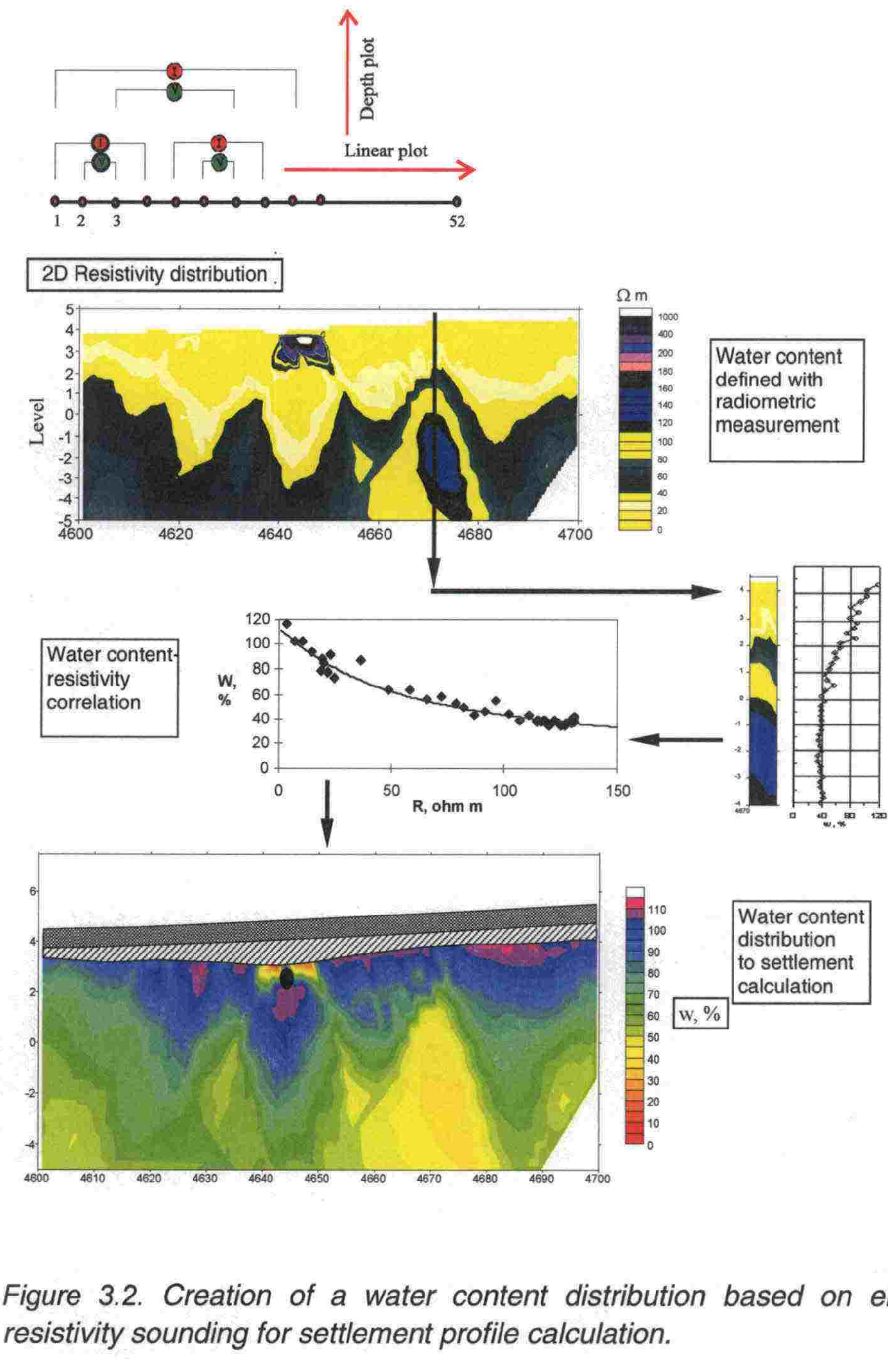

Figure 3.2. Creation of a water content distribution based on electrical resistivity sounding for settlement profile calculation.

3.3 Calculation of the settlement profile

Calculation of a road's settlement profile and obtainment of initial data together form an iterative process. The water content values are converted into water content-dependent settlement parameters γ', e_0, C_c and k at the center points of the pixels, using either general correlations or site-specific conversions based on oedometer tests. They are then further converted into tangent modulus procedure parameters m_1, β_1 (in a normally consolidated area) and a consolidation coefficient c_v (in a normally consolidated area) used in the TSARPIX program. The program defines the corresponding parameters of the overconsolidated area on the basis of entered default values.

In the tentative settlement calculation, general correlations (C_c-w, k-w, c_v-w) defined for homogenous materials can be applied in the water content tomography, and thereby form the first version of a soil structure model of the site. The tentative settlement calculation made on the basis of the first soil structure model is performed to determine the settlement sensitivity of the ground (calculation of settlement potential) and to guide the location of further investigations without all the information that significantly affects the amount of settlement.

To calculate more accurate time-related settlement, information on the consolidation state of the soil layers (OCR or σ_c) and the consolidation coefficient (c_v) or hydraulic conductivity (k) of the soil layers, which determines settlement over time. This information, as well as information on the location of hydraulically conductive layers, is obtained from additional soil investigations, made on the basis of the results of the tentative settlement calculation, and laboratory tests. Supplementary soil investigations include CPTU tests, sampling, vane shear test and location of water conducting layers. On the basis of the results of the oedometer tests, the general water content – compressibility index correlation used in the tentative settlement calculation is corrected for each site.

Soil structure model version 2 is created based on information obtained from the supplementary soil investigations. This soil structure model is used to calculate settlement as a function of time and location. The typical settlement calculation intervals are 1, 5, 10, 15, 20 and 30 years.

3.4 Selection of a road's foundation

Calculation of a road's settlement profile based on electrical resistivity sounding is done for sections of the road line that are critical from the standpoint of settlement. A road foundation or a ground improvement method is selected for areas whose allowed settlement and settlement differences are exceeded during the period of use (e.g.30 years) /2/. Road foundations limit settlement to an acceptable level. In some cases uneven settlements may be significant already before 30 years has passed. If necessary, it is possible to determine the values that define evenness and also determine the impact of settlement on the life span of the road already in earlier phases.

Suitable road foundation methods are selected for areas of the road where the allowed total settlement is exceeded during the period of use (basic period 30 years). Settlement that is less than 200 mm can usually be taken care of with transition structures. For settlement between these two values, the need to limit settlement is considered in each case separately. Transition structures are located in places where changes in inclination of road's surface would be too great. The need for transition structures is assessed on the basis of driving speed according to the principles and criteria presented in the TPPT design system /2/.

3.5 Example

The example site is Route 7 from Hamina to Virolahti, where extensive field measurements were made in 1998-99, and based on the measurements and existing history data, studies were made to determine the structural condition of the road. The field investigations and measurements included electrical resistivity soundings and radiometric soundings at four sites (length about 500 m each). One of these sites (site 2) was chosen as a test site for settlement calculation. The road was built in the 1960s. The results of the study are presented in reference /1/.

3.5.1 Ground resistivity soundings and the water content

The length of site 2 selected for settlement calculation is approximately 400 m. The results of the electrical resistivity soundings performed at the site in 1999, with the different measured sections topographically corrected, combined and recorrected using the RAIPIX program, are presented in figure 3.3. The largest resistance values have been removed in the figure (white areas).

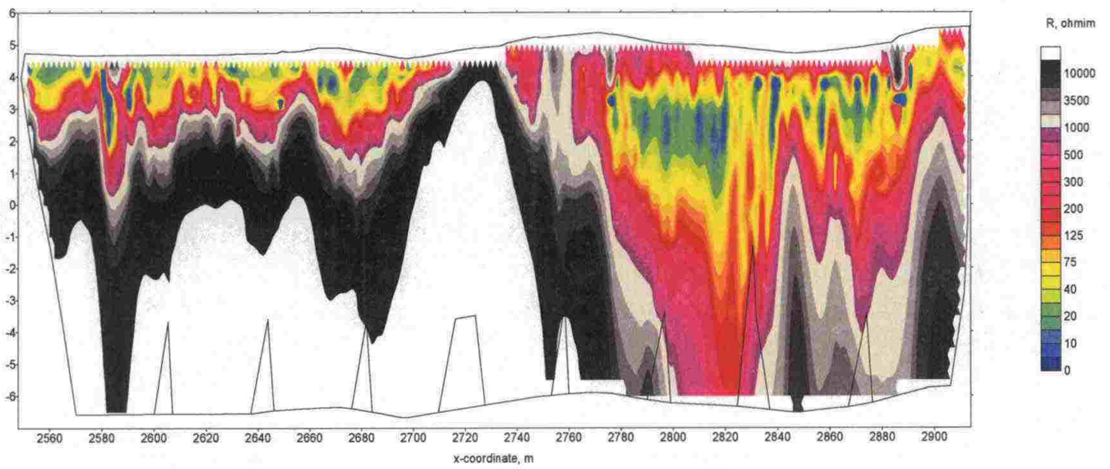

Figure 3.3. Route 7. Corrected results of electrical resistivity soundings of site 2.

The resistance correlation needed for water content correlation was formed by examining the correlations from all four partial sections. The correlations of the partial sections are presented in figure 3.4. From the figure it can be seen that the correlation in site 1 clearly differs from that of the other sites. The final resistance-water content correlation was thus formed on the basis of the observations from sites 2, 3 and 4. This correlation is presented in the figure with a heavy line ("combination").

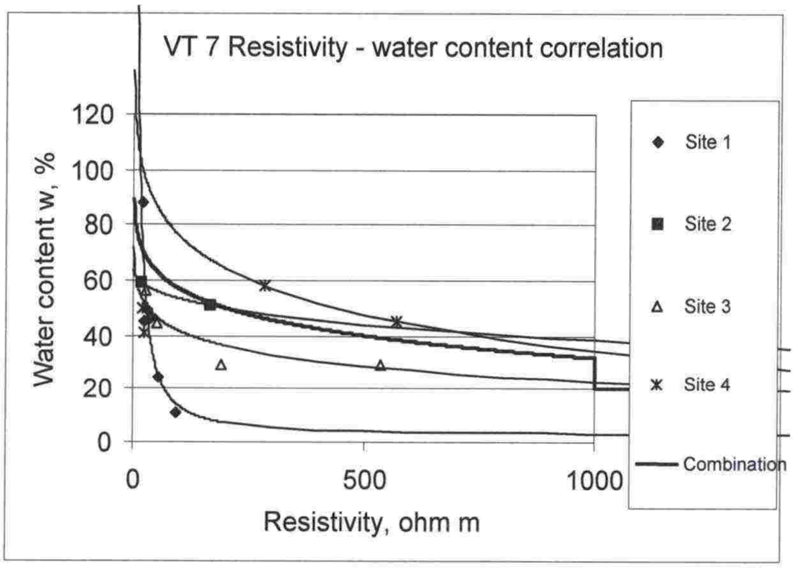

Figure 3.4. Route 7. Specific resistance – water content correlations of four settlement sites and the function (combination) used in water content conversion based on the correlations (see text).

The equation used in the water content conversion is

$$w = -206.831+(3476.955/(10.71804+R^{0.19522}))$$

In performing the water content conversion, the water content of resistance values R > 1000 Ωm was defined as w = 20 %.

The water content distribution obtained from the resistance distribution using the above mentioned equation is presented in figure 3.5. The drawing was created with the Surfer-program according to the principles given in reference /3/. The figure also shows how the grade line and the ground surfaces, which are used as initial data in the settlement calculation, are situated in the measurement results. The ground water level is not shown in the figure. The ground water level used in the calculations was -1.0 m from the surface of the ground.

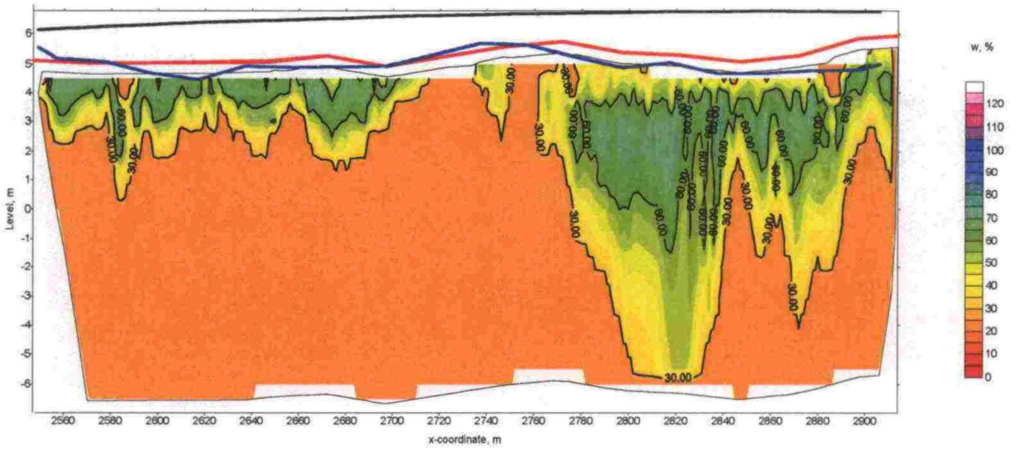

Figure 3.5. Route 7. Water content distribution, planned grade line elevation (black line) and ground surface of site 2 at the measurement line (red line) and the road (blue line).

3.5.2 Settlement profile calculated using the TSARPIX program

The site's load data were created at 1,0 m intervals using the TSARPIX program. This interval was also used in calculating settlement. The settlement profile obtained using the TSARPIX program is presented in figure 3.6. Settlement was calculated for 30 years. It can be seen from the figure that the settlement takes place almost completely during the first 10 years.

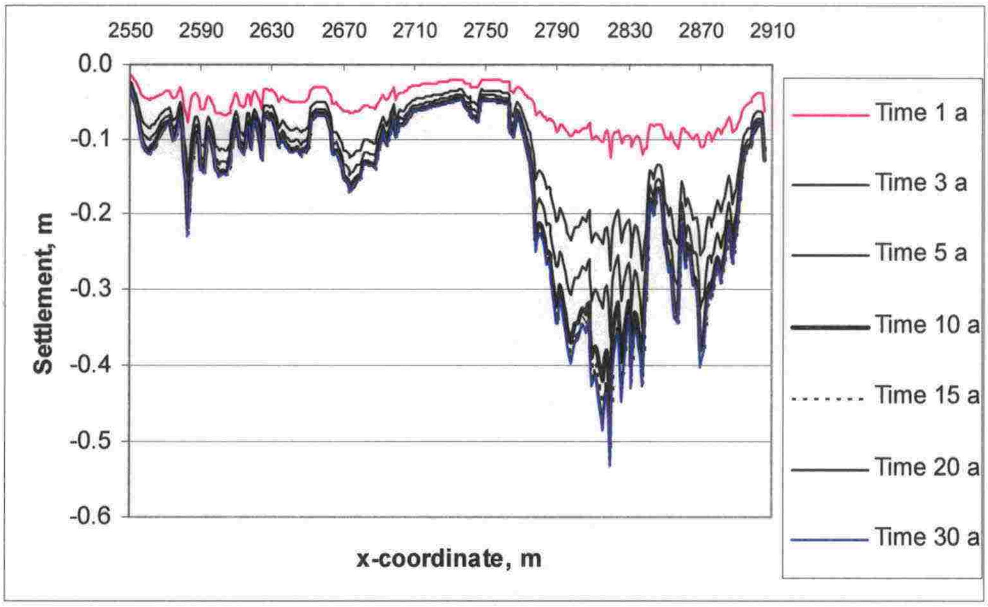

Figure 3.6. Route 7. Settlement of site 2 at 1.0 m intervals, calculated at different times using the TSARPIX program.

3.5.3 Comparison of calculated and observed settlement

To compare the calculated and observed settlement, a 30 m moving average was specified for the settlements calculated at 1.0 m intervals using the TSARPIX program. To define "detected settlement" of the 40-year-old site, it was necessary to take into consideration conversions in the elevation systems, uplift of the soil surface, corrections resulting from estimated paving of the road and phase shifts resulting from coordinate system problems. The site's settlement with these factors corrected is presented in figure 3.7 along with the evened settlements calculated using the TSARPIX program.

It can be seen from figure 3.7 that the settlement obtained from the measurements corresponds quite well to the calculated settlement. The greatest deviations are noticeable in the beginning end at mileage posts 2550...2600, and at mileage posts 2660...2720 and 2840...2900. The settlement at the beginning of the section, where the actual settlement is greater than the calculated settlement, is difficult to explain. However, based on their similarity shape to the calculated settlement and their thickness, the other two areas of deviation can most likely be explained by maintenance measures (re-paving) done to the road.

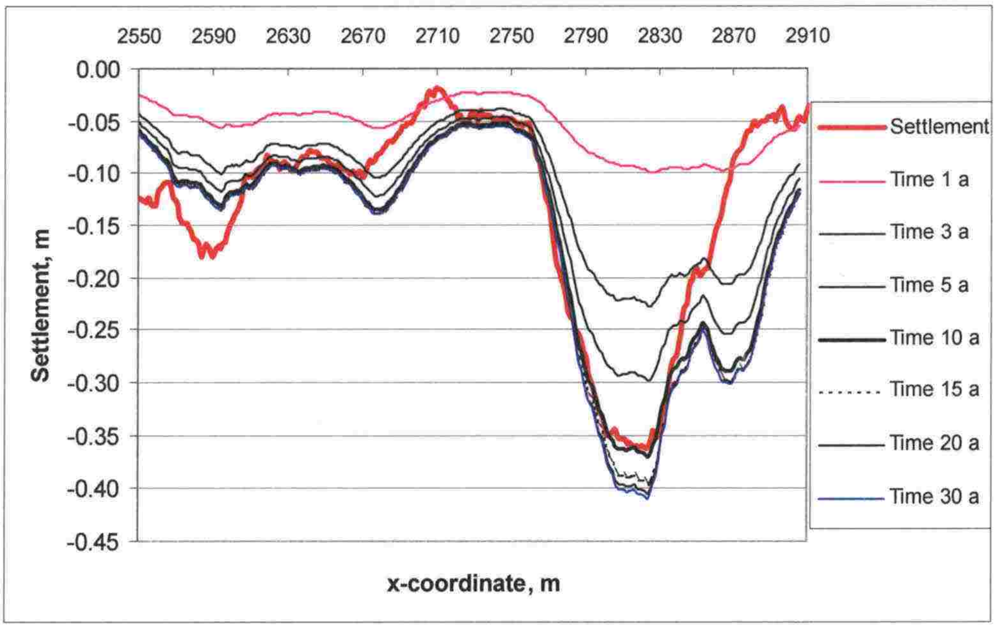

Figure 3.7. Route 7. Settlement of site 2 and settlement calculated using the TSARPIX program and evened using a 30 m moving average.

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4 FROST-RESISTANT STRUCTURES AND FROST DESIGN OF THE PAVEMENT

Frost heaving of frost-susceptible ground causes even or uneven frost heave in the surface of the road. When the frost thaws, the road surface usually returns to its original level. Frost-resistant design of the pavement and, if necessary, frost protection prevent undesirable frost heave and damages in the road surface.

Softening of the subsoil during thawing affects the durability of the structure, primarily through the reduced load-bearing capacity of the subsoil. During the thawing phase the upper part of the unbound structure also loosens, as the structure mainly thaws from the top down and frost heave doesn't retreat until the very end of the thawing period. The undesirable effects of softening can be eliminated by selecting the right materials for the structures and by constructing good drainage.

Thawing begins at the road surface and progresses in proportion to the accumulated sum of degrees of temperature. Depending on the depth of the frost, the period of compression of soil layers softened by thawing lasts 2-4 weeks after the frost has thawed (figure 4.1). Recovery of the load-bearing capacity to its summertime level usually lasts 4-6 weeks due to the slow drying of the structural layers. The start of thawing, the local progression of thawing depth and the end of thawing (the beginning and duration of the period of frost damage) can be determined on the basis of the sum of degrees of temperature of the road surface, while taking into consideration the local frost depth corresponding to the level of wintertime freezing temperatures.

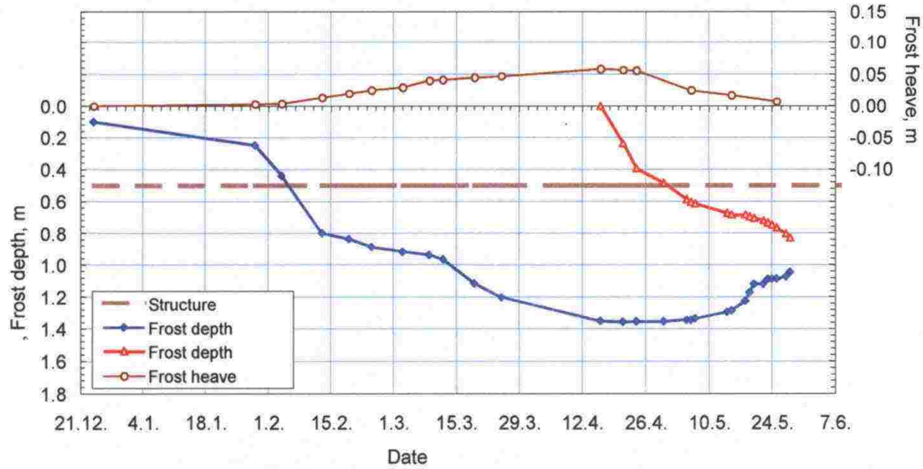


Figure 4.1. Example of the progression of frost depth and thawing over time /3/.

4.1 Development of frost-resistant pavements

Attempts to prevent the undesirable impact of frost on road traffic and frost damage have traditionally primarily consisted of thick, unbound structural layers of frost-resistant materials and adding frost protection, if necessary, to ensure prevention. The design of the structures has mainly been based on the selection of materials and the choice of layer thickness based on experience.

In controlling the undesirable effects of frost according to the TPPT design system, the fundamental principle is that the allowed frost heave is the design criterion of the pavement and the design itself is based on local conditions. The design takes into consideration the material's thermal characteristics, on which basis the thickness of the structural layer is determined. This expands the possibilities of using less-used materials, e.g. by-products in the structural layers of a road.

The following frost-resistant structures and materials suitable for frost protection have been studied at test construction sites in the TPPT program /1/

- Sod peat as a frost insulation
- Stabilization and homogenization of the substructure
- Reinforcement mesh in the pavement
- Exchange of mass
- Blast-furnace sand/slag structure
- Steel profile in the pavement

The test sites have been lower class roads in need of repair due to frost damage (cracks and unevenness, insufficient load-bearing capacity), which nevertheless carry a significant amount of heavy traffic. A traditional, commonly used (reference) structure was also constructed alongside the test structures at the sites. At certain sites work methods were also developed. The economy of the test structures in relation to traditional structures was not examined because it is very difficult to obtain comparable cost information from the test construction sites. Certain test construction sites belonging to the first three categories mentioned above and results obtained from the sites are briefly described in the following.

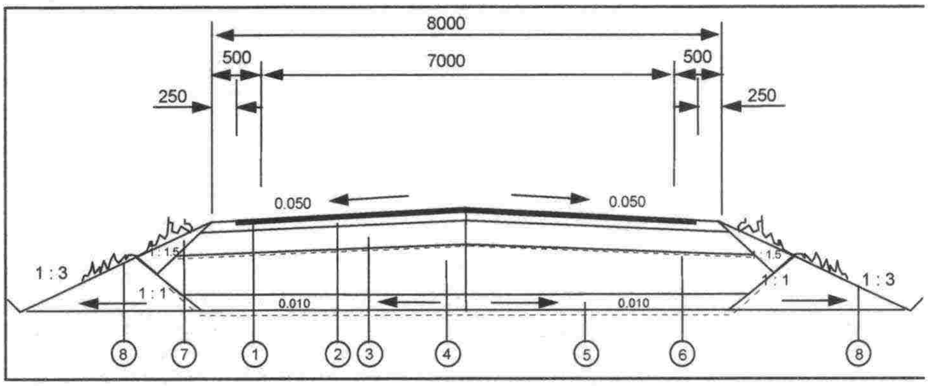
The performance of the test construction sites was monitored for up to six years. The temperature (freezing index) of the monitored winters varied from a mild winter that occurs once in ten years to a winter that was slightly colder than average.

4.1.1 Sod peat structures

The objective of sod peat structures was to achieve sufficient frost resistance by preventing the frost-susceptible layer of the substructure from freezing, undesirable frost heaving and loss of strength during the thawing phase. The thickness of the sod peat layer was to control the amount of frost heave and the capacity of the thawing phase.

At most sites the sod peat layer was constructed in a longitudinal wedge shape, because the intent was to monitor the effect of the thickness of the sod peat insulation on frost damage and the amount of frost heave. The subsoil at all the sites was frost-susceptible, either moraine, silt or silty sand.

For example, the thickness of the sod peat layer at the Ranua site /8/ was 300 mm at its thinnest and 600 mm at its thickest. The design freezing index at Ranua is about F10 = 42,000 Kh. To even undesired differences in frost heave, a transition wedge was constructed at both ends of the sod peat structure. The sod peat structure was constructed as a so-called soil box in which there is 200 mm of sand below the sod peat layer that prevents the capillary rise of water from the subsoil and works as an underground drainage layer. To prevent intermixing of the layers, a geotextile was installed on the top surface of the sod peat layer and below the bottom surface of the filter sand layer. The base layer was an 80 mm Remix-stabilized layer and the subbase was a 400 mm layer of crushed aggregate. The wearing course consisted of 30 mm of SAC (figure 4.2).



1. Wearing course; SAC	30 mm
2. Base; Remixer stabilization	80 mm
3. Subbase	400 mm
4. Sod peat	300 – 600 mm
5. Drainage layer; Sand	200 mm
6. Geotextile	
7. Slope	
8. Lining	

Figure 4.2. Sod peat structure of the Ranua test construction site, cross section.

The following results have been obtained from long-term observations:

- The volumetric water content increased significantly for the main part over six years. At its greatest the volumetric water content in the thawed state was between 44.7 – 72.5 vol.-% at the end of the monitored period, while immediately after construction it varied between 13.7 – 31.2 vol.-%.

- The thermal conductivity of the sod peat structures increased quite clearly during the monitoring period (to 0.40 – 1.19 W/Km), but did not reach the values estimated in the design (thermal conductivity when thawed 0.6 W/Km and when frozen 1.2 W/Km). During the first monitored winter the thermal conductivity of the frozen sod peat varied between 0.18 – 0.24 W/Km. The increase in thermal conductivity was also detected as increased frost penetration.

- Compression was observed after construction, especially in the thick sod peat layers. The crossfall designed into the structures was preserved relatively well, though.

- The total thickness of unbound layers in a sod peat structure should be much greater than what was used in the test structures. Load-bearing capacities measured from on surface of the road were low. Because the modulus of rigidity of sod peat and the unbound layers above it are small, the critical stresses affecting the sod peat are great and therefore the load-bearing capacity of the road is poor.

The following remarks can be given of the usefulness of sod peat as a frost-insulating material

- Because sod peat as a structure is relatively compressible, sod peat is suitable for a road that doesn't have heavy traffic. Also, sod peat comes into question primarily where the material is economically available.

- The design is based primarily on practical experience. The structure requires a thick subbase layer. The thickness of these layers depends above all on the thickness and rigidity of the bound pavement layers and the thickness of the sod peat layer itself /12/.

- Sod peat must be drained so that it remains dry all year round.

4.1.2 Treatment of the frost heaving layers and subsoil

Homogenization and stabilization of the subsoil was studied in test structures (Kiuruvesi, /5/), where the uppermost layers of the road were renewed and the moraine (till) substructure was homogenized and the top surface of the substructure was stabilized with different amounts of cement (figure 4.3). The old structural layers of the road were removed (the entire road or one lane at a time) and then the old substructure and subsoil were raked with an excavator's bucket to remove stones as deeply as 2 m below the new grade line of the road (non-frost heaving depth). Mixing was performed by digging and filling. A prescribed amount of binding material was spread on the surface of the raked and mixed layer and mixed into the raked layer to a depth of either 0.5 m or 1.0 m with either an ordinary or a specially structured excavator bucket. The mixed layer was compacted with a heavy vibrating plate.

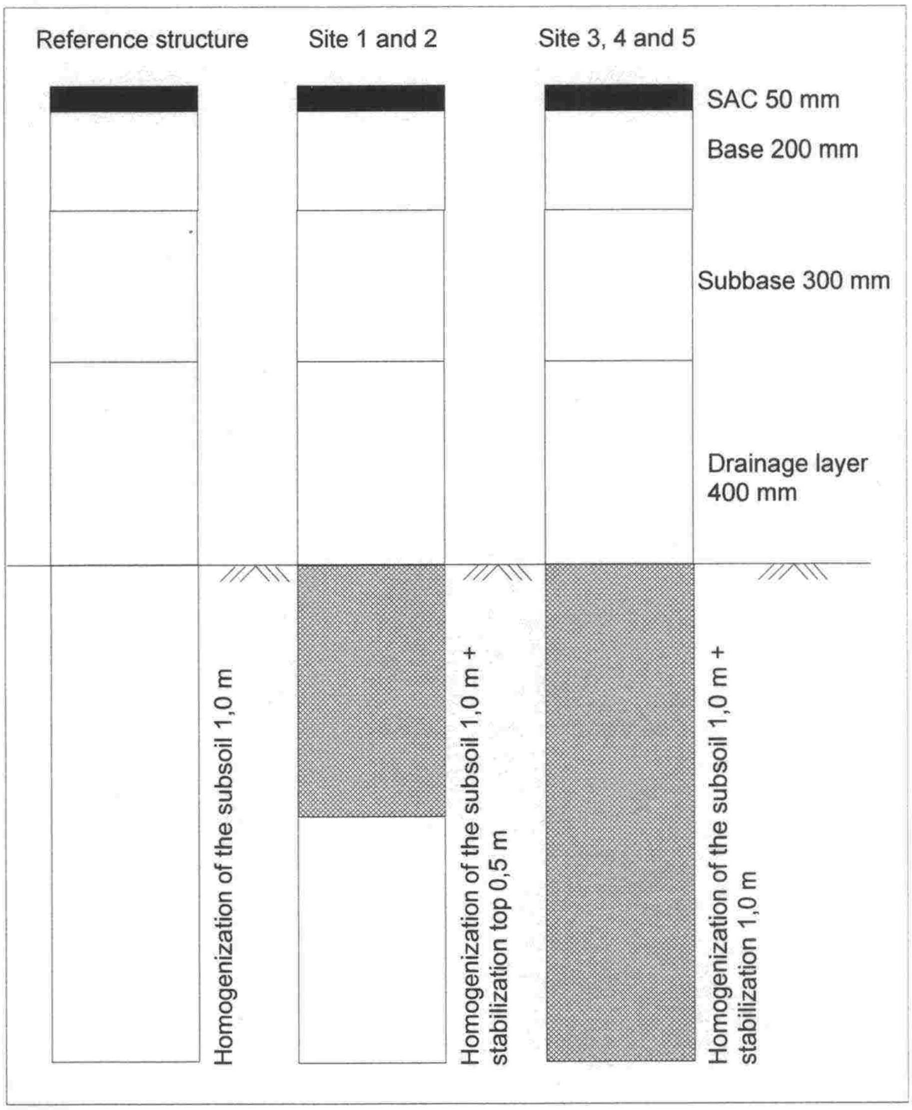
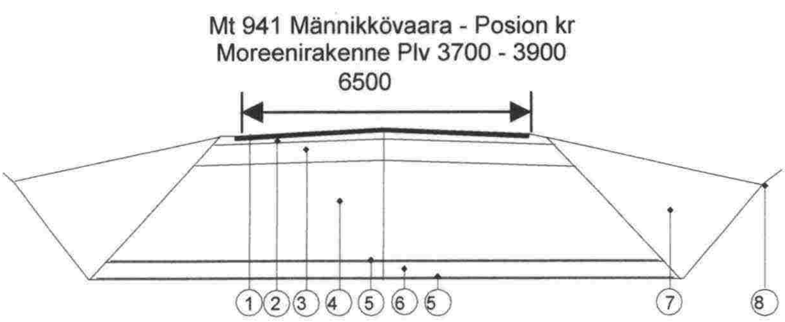


Figure 4.3. Structural layers and procedures in the homogenization sites of the substructure, Kiuruvesi (F10 = 43,000 Kh).

In another site (Salahmi, /6/), the moraine substructure was homogenized and stabilized with a Finnstabi™ + lime mixture. A detour was built around the site so the entire old road could be treated as one roadwork site. The thickness of the stabilized layer was 0.5 – 0.6 – 0.7 m and the target density was 90 % and 95 %. The working technique was the same as at the Kiuruvesi site except that the binding material was mixed into the substructure with a "Maamyyrä ™" drum mixer. After mixing the layer was compacted with a vibrating roller. The reference part of the test structure was constructed so that the subsoil was homogenized to a depth of 2 meters from the grade line of the road. The thickness of the new pavement was 950 mm.

The objective in the Männikkövaara test structure /9/ was to achieve sufficient frost resistance and control frost heave by improving drainage of the moraine structure with an underground drainage layer of gravel under the layer of moraine. More effective drainage of the moraine structure also decreases the amount and duration of softening during thawing. The design freezing index (F10) at the site is 51,000 Kh.

In the moraine structure the pavement and subsoil (sandy till of relatively homogen quality) of the old road were mixed together to a depth of about 2 meters below the grade line of the road. The granularity of the mixture of pavement and moraine was equivalent to sandy moraine. The moraine mixture was used to construct a road embankment on top of a gravel layer that functions as an underground drainage layer and blocks capillary rise. The layers of the moraine structure from the top down were SAC 40 mm, Remix-stabilization 70 mm, base and subbase 300 mm, moraine embankment 1,400 mm, geotextile, gravel 200 mm and geotextile (figure4.4).



1. Wearing course; SAC	40 mm
2. Base; Remixer stabilization	70 mm
3. Subbase	300 mm
4. Moraine (till) embankment	1400 mm
5. Geotextile	
6. Drainage layer; Gravel	200 mm
7. Slope	
8. Lining	

Figure 4.4. Cross section of a mixed moraine structure.

The following results have been obtained from longterm observations:

- Homogenization of the subsoil and renewal of the structural layers decreased frost heave around 35 % (reference structure) compared to maximum frost heave in the old structure (200-250 mm). Frost heave increased during the five monitored winters compared to frost heave during the first year.
- Cement stabilization of the substructure together with homogenization of the subsoil and renewal of the structural layers decreased frost heave around 50 % compared to frost heave in the old structure. The thicker the stabilized layer was, the smaller the frost heave was.
- Sufficiently deep stabilization was important in preventing frost heave.
- Damages observed in the test structures most likely were caused by insufficient compaction of the layers and homogenized / stabilized subsoil during construction.
- Although frost heave was quite considerable in a mixed moraine structure, differences in frost heave between the center line and edges of the road were very small.

The following remarks can be given of the usefulness of homogenization, stabilization and a mixed moraine structure

- Homogenization of the frost-susceptible substructure and subsoil of an old road is a useful way to prevent frost damage. Stabilization increases the frost resistance of the layers.
- There is no available technically efficient and economical method for homogenizing thick layers and mixing binding material.
- A structure consisting of a mix of the structural layers of the old road and the subsoil, where an underground drainage layer is constructed underneath the mixed layer to improve drainage, is rarely competitive compared to other methods (such as replacement of mass) and its technical performance is questionable.

4.1.3 Reinforcement meshes

The purpose of reinforcement meshes is to prevent damage to the pavement caused by the combined effect of frost heave and traffic load by strengthening the structure with steel meshes. The issues studied at the test construction sites were:

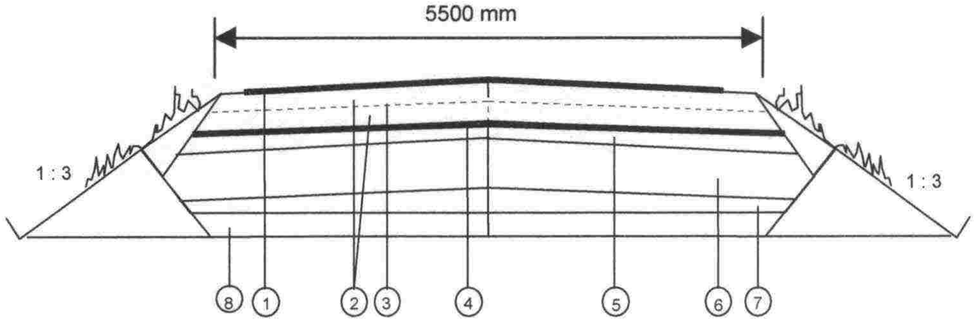
- Sensible applications of using a steel mesh structure, the distance between reinforcement mesh elements, their location and layer thicknesses.
- Development of design method (layer thickness and materials, steel sizes, interpositions).

At the Ranua site /8/ a steel mesh was installed on top of Remixer stabilization before spreading a leveling mix. The mesh extended to the edges of the paved area. Observations indicated that the pavement structure of the road lifted as a slab, and there were no typical longitudinal cracks in the middle of the road caused by considerable frost heaving of the subsoil.

A survey of damage conducted before the planning of the Temmes site /7/ showed depressions at the edges of the road and longitudinal, cross and alligator cracking. The pavement had been patched in several places. Once every ten years the freezing index at Temmes is F10= 42,300 Kh. Several test sections that differed with respect to the old road structure and the location of the mesh were implemented in Temmes. Two exemplary test sections are described in the following.

The steel mesh that was used is B 500 K 7/5-150/200 F30. The length of the element (in the longitudinal direction of the road) is 2,350 m, the width corresponds to the width of the road, the diameter of the steel wires 5 mm in the longitudinal direction and 7 mm in the cross direction. The distance between the wires is 150 mm in the longitudinal direction and 200 mm in the cross direction. First, five steel mesh elements were placed 20 cm apart. Then five elements were placed 40 cm apart. This increase in spacing was repeated until the last mesh elements were 235 cm apart. The steel mesh elements in a reference structure were continuous. Construction of the test construction sites was started by removing the oil gravel pavement of the old structure. At one site the mesh was placed on top of the old base layer (hkSrMr - sandy gravel moraine) and covered with 200 mm of crushed aggregate. The final distance of the steel mesh from the top surface of the base layer varies between 100...190 mm, with an average distance of 160 mm. At the other site, where the old road structure included a soil cement layer, the soil cement layer was covered with a 100 mm layer of crushed aggregate on which the mesh was placed. About 300 mm of crushed aggregate was placed on top of the mesh. The final distance of the steel mesh from the top surface of the base layer varied between 270...370 mm, with an average distance of 300 mm. The layers of crushed aggregate were compacted well.

The old road structure (soil-cement concrete site) under the layer of crushed aggregate is shown in figure 4.5. The wearing course on the new road was a 40 mm SAC layer.



1. Wearing course; SAC	30 mm
2. Base; crushed aggregate	400 mm
3. Steel mesh 5/7 k200	
4. Oil gravel; removed	30 mm
5. Soil-cement	120 mm
6., 7., and 8; old pavement structure	550 mm

Figure 4.5. Cross section of the test construction site at Temmes (soil-cement structure)

The following results have been obtained from long-term observations:

- Due to the effect of the steel mesh, the differences in frost heave between the center line and edges of the road were very small, with a maximum of 10-15 mm.

- Although maximum frost heave was nearly 300 mm, no cracking appeared in the center line of the road. The structure made rigid with the mesh lifted as an even slab.

- The width of longitudinal cracks appearing at the edges of the road in conjunction with minor frost heave (less than 100 mm) did not depend on the manner of installation (distance between elements 0.2 - 2.35 m). With greater frost heave the amount of longitudinal cracks at the edges of the road and the seriousness of the damage increased as the level of frost heave and the distance between mesh elements grew.

- Correspondingly, the amount of damage increased from the edge of the road toward the center as the level of frost heave and the distance between mesh elements grew.

- No ruts were formed in the pavement at the reinforced sites during the six-year monitoring period.

The use of steel meshes in the road structure is covered in reference /13/. An ongoing EU project "REFLEX" dealing with the use of steel meshes in the road structure is developing the design of this type of structure on the basis of an extensive test road program and results of HVS tests. A handbook will be compiled based on the results. The Finnish Road Administration is participating in the project.

The following practical views on the use of steel meshes in the prevention of frost damage can be presented:

- A reinforcement mesh installed in the pavement prevents longitudinal and diagonal cracks of the pavement.
- A reinforcement mesh placed in the pavement does not prevent the formation of cross cracks because the joints in the mesh elements do not withstand tensile stress, and the reinforcement mesh does not prevent the formation of longitudinal cracks in the shoulder at the edge of the pavement.
- A reinforcement mesh placed in the pavement evens differences in frost heave in the cross direction, but at the same time increases cracking in the road's shoulder.
- A sufficiently wide reinforcement mesh placed in the unbound base / subbase reduces the amount of longitudinal cracking and deformation of the layers above the reinforcement mesh.
- A reinforcement mesh placed in the unbound base/subbase increases the load-bearing capacity and decreases rut formation in the subsoil, especially during the thawing phase.
- A reinforcement mesh in the pavement does not decrease frost heave. Frost heave has to be controlled using frost-resistant designing.

4.2 Frost design of the pavement

The fundamental principles of designing frost protection and frost-resistant pavement design are

- in areas with frost-susceptible subsoil, frost heave of the road surface does not exceed a value that is considered undesirable from the standpoint of structural performance and evenness of the road, and
- the probability of the occurrence of damaging frost heave and the extent and repetition of unevenness are under control.

In frost-resistant design /4/, frost heave of the road surface is calculated for the designed structure using a selected winter design freezing index. The calculation requires information about the frost heave and thermal characteristics of the subsoil and the layer thickness and thermal characteristics of the frost-resistant pavement. In frost-resistant design the frost heave characteristics of the subsoil and substructure are depicted in the TPPT design system by a frost coefficient (SP, segregation potential).

4.2.1 Design criteria

The amount of frost heave allowed in a road is specified on the basis of the risk of damage to the pavement. Frost heave, which usually is uneven, also causes unevenness in the pavement, which increases as frost heave increases. Unevenness may change the inclination of the surface and also make it difficult to keep the surface dry. Table 4.1 presents the limiting frost heave of a road structure (not reinforced structures).

Table 4.1. Limiting frost heave on different types of roads and structures (TPPT values, /2/).

Structure/ pavement	Design freezing index F, Kh	Max. frost heave, mm	Max. difference of frost heave, o/oo
Motorway	F_{10}	30	5
Main road	F_{10}	50	7
Local road	F_{10}	100	10
Other traffic area			
Stone pavem.	F_{10}	50	6
Asphalt	F_{10}	100	10
Gravel	F_{10}	150	15-20

4.2.2 Determination of the segregation potential

The segregation potential (SP) describes the degree of frost heave in the subsoil. It refers to the ratio between the rate of frost heave and the temperature gradient affecting at the frost line. The temperature gradient is defined as the ratio between the average surface temperature and the frost depth. The SP is needed in designing the frost protection of the pavement, as continuous data along the new or repaired road line.

The SP can be determined on the basis of either

- back-calculating from frost heave observations of an existing road using approximate equations or curves (recommended in conjunction with designing the repair of frost damage) or
- frost heave tests in laboratory or
- approximate estimates of the subsoil on the basis of classification characteristics of the subsoil.

Back-calculation requires information about the temperature of the observed winter, the pavement, the quality of the subsoil and the amount of frost heave of the surface of the road. The SP can be calculated using the approximate equation (4.1) /3/.

$$SP = \frac{50h}{10\sqrt{F} - z_o} - 1{,}8 \qquad (4.1)$$

where SP is the segregation potential, mm^2/Kh
z_o the thickness of the surface structure, mm
h frost heave, mm
F freezing index, Kh

4.2.3 Calculating frost heave

Frost depth can be calculated using Watzinger's method. This method can be used in preliminary frost-resistant design of the pavement and in back-calculation of the SP. It can be used to estimate frost heave along the center line and edges of the road if the reduction of temperature at the edge of the road is known. The SSR-model /10/ is used in compiling the design curves of the TPPT design system. An Excel spreadsheet program "SSR.XLS" for frost heave design has been made of the model (VTT).

4.2.4 Frost protection using frost-resistant mineral soil

If the SP of the substructure and the design freezing index of the site are known, the thickness of the frost-resistant layer of a road structure can be estimated on the basis of observed frost heave. Figure 4.6 presents the relationship between frost heave, the SP of the subsoil and the thickness of the road structure at a design freezing index of F = 40,000 Kh. It is assumed the subsoil is frost-susceptible silt with a dry density of 1.6 t/m^3 and a water content of 25 %. In TPPT's method description of frost design the corresponding curves are presented at 5000 Kh intervals beginning at a freezing index of F=20,000 Kh and ending at F=60,000 Kh.

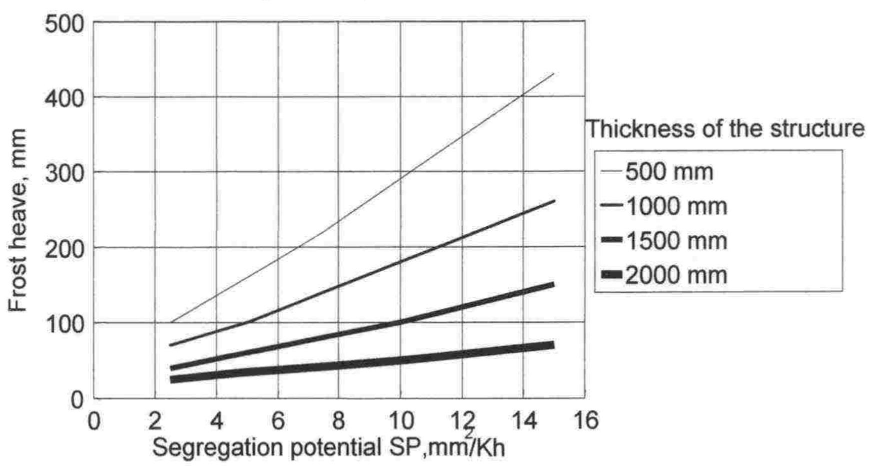


Figure 4.6. Specification of the necessary thickness of a frost-resistant road structure with the help of frost heave observations and design freezing index (e.g., F = 40,000 Kh).

4.2.5 Design of a frost-insulated structure

Frost protection of a road can also be made using frost insulation. The frost insulation may be polystyrene (EPS or XPS), expanded clay, or other thermal insulating materials approved for use in road structures. The materials need to have sufficient load-bearing capacity and their stability under various influences (such as oil spills) must be structurally ensured.

The necessary thermal resistance of the frost insulation layer can be estimated using curves such as those shown in figure 4.7. In TPPT's method description of frost design the corresponding curves are presented at 5000 Kh intervals beginning at a freezing index of F=20,000 Kh and ending at F=60,000 Kh. In plotting the curves it has been assumed that there is at least 700 mm of frost-resistant layers on top of the insulation and a drainage layer at least 300 mm thick underneath. The plotted curves can also be used in the preliminary design of the structure, whereupon the freezing index is the freezing index of the design winter. The thickness of the frost insulation is specified on the basis of the thermal resistance and thermal conductivity of the insulation material.

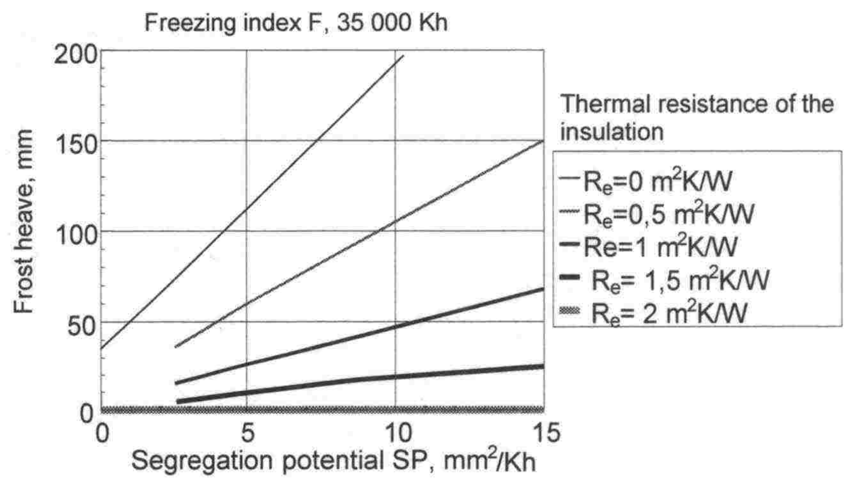


Figure 4.7. Specification of the necessary thermal resistance of frost insulation on the basis of frost heave and the segregation potential (SP) when the freezing index is 35,000 Kh.

In specifying the design values of thermal conductivity and load capacity of materials used in frost insulation of buildings /11/, it is required that they preserv the specified design values for a period of 50 years. During this 50-year period the thermal values of the insulation weaken from those of "factory-fresh materials", especially from the effect of moisture. The design thermal conductivity values include a calculated 10 % margin to account for the method of installation, compression of the frost insulation and other unforeseen conditions and factors. The product specifications usually give the material's design values in dry conditions.

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5 FATIGUE DESIGN AND PAVEMENTS

5.1 Design and development of pavements

In Finland, pavements have traditionally consisted of a thin asphalt courses and thick unbound layers. However, we also have an abundance of thin structures with a wearing course consisting of a relatively flexible layer of asphalt. These different types of structures behave differently under loading, which should be taken into consideration in development work and when specifying design parameters.

Fatigue design of thick structures is based on a mechanistic-empirical model in which responses calculated according to an analytical theory are used to estimate pavement damage. Damage is described using fatigue models in which material durability is usually expressed as strain in relation to the loading repetitions. Because the fatigue characteristics of different materials differ from each other, each type of material should have its own fatigue model.

Design based on fatigue is suitable for use with thick road structures whose damage mechanism primarily involves structural layer fatigue. The fatigue model as such is not very suitable for designing thin road structures. Traffic load may inflict high stresses on thin road structures, thereby causing permanent deformation in the unbound layers and in the subgrade, for which reason the primary cause of damage is structural deformation.

The TPPT design system presents the procedures and methods with which a road structure can be designed using site-specific data and parameters (traffic, climate, subgrade, materials, old structures). The parameters of structure materials and subgrade are primarily specified on the basis of laboratory tests or measurements and studies made in the field. "Methodology descriptions" dealing with acquirement of initial data for planning and design are an essential part of the design system.

TPPT development work concerning pavements concentrated on asphalt layers, bitumen stabilized structures and composite structures. New economical possibilities were investigated that involve refining local, low-quality materials, such as moraine, to make them suitable for use in road structures.

5.2 Strain and stress on the pavement

A road is designed to sustain loads caused by a predicted volume of traffic. The types and quantities of heavy vehicles are significant from the standpoint of road structure stresses. In addition to traffic volume, another important issue is vehicle axle weight. Passenger cars only cause rutting and thinning of the pavement surface as a result of wear from studded tyres /15/.

Vehicle weights have been studied by measuring axle weights individually. Extensive manual weighings of vehicles are conducted every 10-15 years. The most recent axle mass study was done in 1998, when a total of 3587 vehicles were weighed at 48 different weigh stations. Considerable information about vehicle structures, transport routes and types of loads was gathered in conjunction with the weighing /15/. The previous study was conducted in 1986.

Not only has traffic volume increased, but in recent years there have also been definite changes, especially in heavy vehicles. The number of vehicles with a trailer has clearly risen and their total weight has grown noticeably. Also, an increasing amount of dual wheels are also being replaced by super single wheels. This trend will most likely continue in the future, and traffic planners, as well as pavement designers need to be aware of the changes.

Fatigue design is done according to the traffic loads exerted on a structure during a design period. Traffic loads are taken into consideration in designing, either as the incidence of load repetitions using an equivalence factor method or by using the axle weight distribution per weight class of heavy vehicles in calculations /15/.

The vehicle equivalence factor method means each vehicle class has its own equivalence factor with which traffic volume in each class is converted to correspond to the number of standard axle (100 kN single-axle, dual wheel) passes of the cross-section of a road. The counts calculated for each vehicle class are added up to obtain the incidence of load repetitions caused by all traffic in terms of standard axles. Multiplying this daily incidence of loads by the length of the period under study gives the cumulative incidence of load repetitions, which is the initial value used in fatigue design.

Vehicle equivalence factors are specified based on the manner in which a road is damaged. The following table 5.1 presents the equivalence factors used to calculate the incidence of loading. Old values are used when dealing with traffic up to 1998 and new values are used from 1999 onward. The new values are calculated from the results of the most recent axle mass study. They predict the situation ten years from now (2009 is the design year), assuming that traffic will mainly consist of newer vehicles in the future.

Table 5.1. Vehicle equivalence factors /18,17,24/.

Vehicle type	Old (1995)		New (predicted for 2009)
	Average	Full	Average
buss	0,4		1,2
truck (without trailer)	0,4	1,5	0,7
semi trailer truck	1,5	3,0	1,7
trailer truck	2,3	4,5	3,2
truck + bus	0,4		0,8
trucks with trailer	2,1		2,8
TOTAL HEAVY	1,3		2,2

The equivalence factors of the trailer truck class is specified so that the value is weighted between the current "semi trailer" and "trailer" coefficients on the basis of information collected from LAM points on main roads in southern Finland.

5.3 Fatigue criteria used in the design of thick pavement

Fatigue design of thick pavements is based on a mechanistic-empirical model in which stresses and strains calculated according to an analytical theory are combined with pavement damage detected in the field. The design system is applicable to hot mix asphalts (AC: asphalt concrete, ACB: base course asphalt concrete and SMA: stone mastic asphalt) with an asphalt layer at least 60...80 mm thick. The method as such is not applicable in the design of cement stabilized structures or otherwise reinforced structures or light (thin) paved roads. The design of light paved roads is covered in chapter 5.6 /14/.

Fatigue design of pavement is based on tension strain at the bottom surface of the asphalt layer, which explains traffic-related damage caused by pavement fatigue.

A fatigue curve specified in the laboratory or with the heavy vehicle simulator (HVS) must always be calibrated with damage observed in the field before it can be used as a design criteria, which is called the fatigue criteria.

5.4 Damage mechanism with pavement over 60...80 mm thick

It is typical of road damage that even on a short section of road, cracks form at different times in several places. This is due to variation in material characteristics, layer thicknesses and subgrade characteristics. Together with different loading features (traffic, climate), this results in a complex road structure system in which crack formation is random by nature /14/.

Once cracks have appeared in the surface of the pavement, a road structure behaves differently than it did before the damage. This is due to discontinuities in the pavement caused by the cracks, whereupon the bound layers no longer function as a load-distributing slab as they did when they were intact.

Modeling of the damage process is divided into three separate parts as shown in the following diagram (Figure 5.1):

1. Modeling the appearance of the first fatigue crack (a crack in a rut) after rehabilitation or paving a structure.
2. Modeling the development of fatigue cracks.
3. Predicting the total damage of a road, damage caused by climate stress.

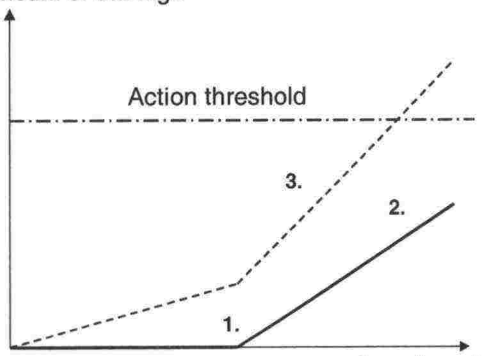

Figure 5.1. Damage process of a road structure. /14/

Models for predicting the start of fatigue damage were developed in the PARIS project /25/. The PARIS project (Performance Analysis of Road Infrastructure) was a common European project including 18 organizations (from 15 countries), which was conducted with partial funding by the European Commission. The project developed road life models that could be used in PMS systems. The data used to develop the models reflects European climate, traffic and material conditions. There were over 800 test roads in 15 countries.

The model used in TPPT design is based on the model of the PARIS project, which was calibrated for Finland's conditions. Modeling was done in cooperation with Sweden's VTI (Statens väg- och transportforsknings-institut) using data from both TPPT's and VTI's test roads. The developed model is a probability model in which damage is explained using information describing the structure and stresses exerted on the structure. The model is based on data from 429 test road sites, of which 50 are in Finland and the remainder are in Sweden. The structures are conventional structures with one, two or three asphalt layers on top of unbound layers. The age of the sites, which is calculated from the previous paving or rehabilitation, varies from 3 to 16 years. The sites have been monitored for research reasons for 9 – 16 years. Traffic volume at the sites (annual incidence of loading) varies between 30,000 and 450,000.

The result obtained was the field-calibrated fatigue criteria of the TPPT reference structure (equation 5.1), based on the strain at the bottom of bound layers, which specifies the start of load durability damage:

$$N_{10} = 10^{7.29-0.00372*(EPS)-5840000*\left(\frac{1}{EPS*N_{10}Y}\right)}, \qquad (5.1)$$

where,

N_{10} =	cumulative number of loads at the time damage starts, 100 kN
EPS=	allowed strain at the bottom bound layer, µm/m
$N_{10}Y$ =	average annual load repetition during the design period, 100 kN

The term at the end of the model, $\left(\frac{1}{EPS*N_{10}Y}\right)$, describes the effect of asphalt aging on the start of road damage. The slower traffic loading is exerted on a road (small N10Y), the less loading the road as a whole sustains, i.e., the effect of aging on damage increases.

The model can be used to calculate the cumulative number of loads from completion of the last paving to the start of damage. The prediction model curves in figure 5.2 are plotted for four different annual average load repetition values (originally traffic volume). The curves are calculated for typical strain levels at the bottom of bound layers.

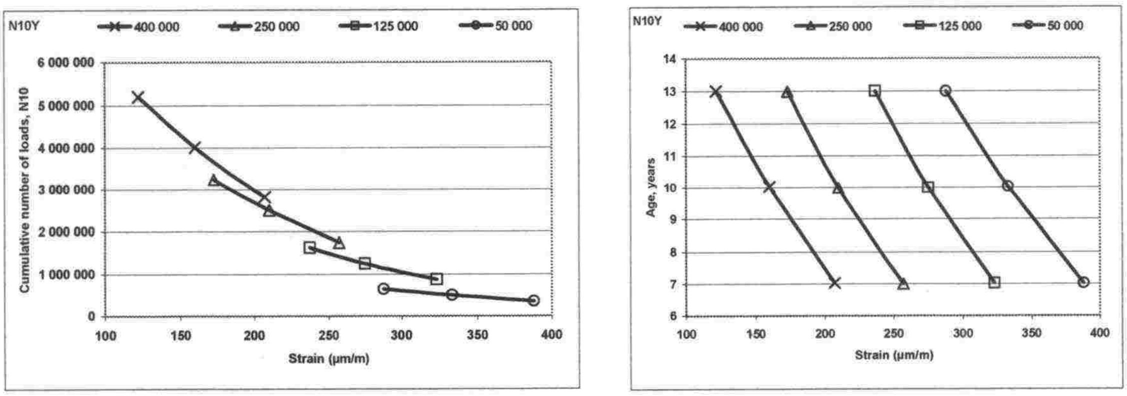

Figure 5.2. TPPT fatigue criteria model curves for four different annual average load repetitions N10Y (traffic volume) in relation to cumulative number of loads and age. /14/

When using the prediction model it is necessary to take into consideration the composition of the data used to develop the model. The model is only applicable in the area covered by the data. The theory behind the modeling also limits the universality of the model. For example, if the loads accumulate too slowly, fatigue used as the basis for the TPPT model is no longer the primary damage mechanism, and pavement damage appears for other reasons.

Because no practical information based on field observations is available on the behavior of new or less frequently used pavement solutions, a fatigue curve must first be specified for the material in a laboratory using a fatigue testing device. Figure 5.3 shows the results of fatigue tests of a few materials. With the help of the fatigue curve and reference material it is possible to specify a relative damage coefficient for the material, which is called a relative "fatigue coefficient". This coefficient is used to calculate the relative durability of the material under field conditions. The specification process includes the following phases:

1. Specify the cumulative *load repetition* for the design period
2. Calculate the *strain* corresponding to the *load repetition* using the field-calibrated fatigue criteria of the reference structure
3. Specify the strain corresponding to the load repetition using the *reference material's laboratory fatigue curve*
4. Specify the strain corresponding to the load repetition using an *alternative material's laboratory fatigue curve*
5. Calculate the *fatigue coefficient* of the laboratory strain
6. Multiply the deformation obtained using the field-calibrated fatigue criteria of the reference structure with the fatigue coefficient of the laboratory strains
7. Continue design using the design procedure of the reference structure

A fatigue test can be used to compare the characteristics of different asphalt masses, while a heavy vehicle simulator (HVS) can be used to compare the behavior of different structures.

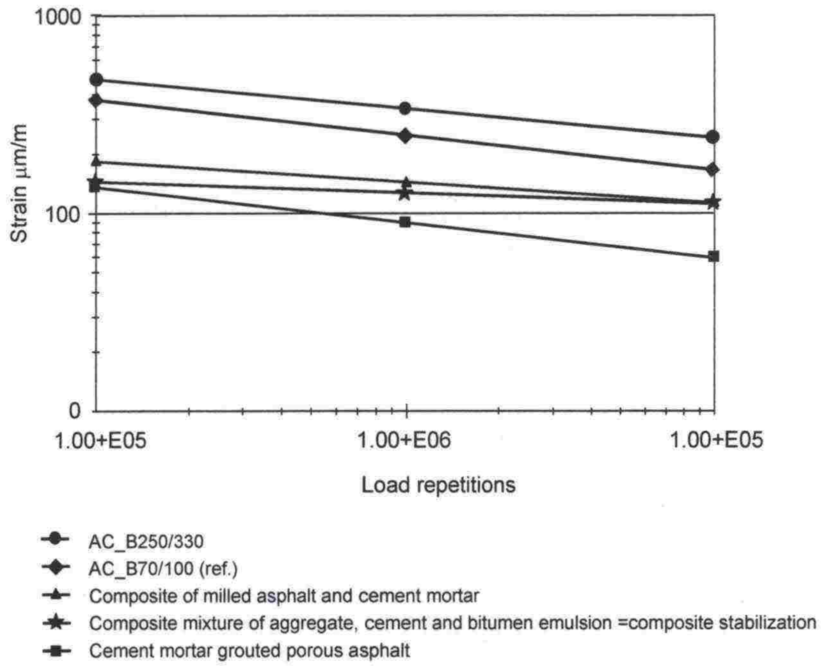

Figure 5.3. Material fatigue curves specified using laboratory fatigue tests (stress control) /14/

5.5 Calculating stresses and strains

Stresses and strains in pavement are calculated with a linear-elastic multi-layer program. Usually, longitudinal tensile strain at the bottom surface of the bound layers of a pavement is calculated. The road structure may also be damaged in other ways (e.g. structural layers are damaged by climatic reasons), but that kind of damaging is not included in fatigue design.

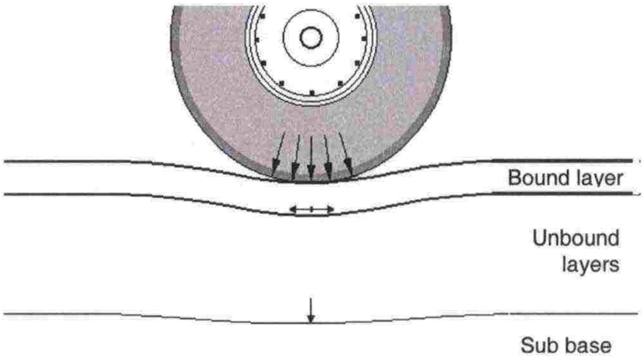

Figure 5.4 Tensile strain at the bottom of pavement caused by wheel load.

The initial data for calculation include structural layer thicknesses, moduli and Poisson's ratios. FWD loading is used as the standard load. The calculation results are stresses, strains and displacements at the calculation points, e.g. under loading at the bottom side of the pavement.

Calculated strains are compared to allowed strains. If calculated strain exceeds allowed strain of the design period, the structure must be made thicker. Correspondingly, if calculated strain is less than allowed, the thickness of the structural layers can be reduced on condition that the resulting thickness is not less than the minimum thickness of frost-resistant design.

Thus, by iteration a final structure is achieved in which the calculated and allowed thicknesses are nearly the same. The principles of this process are illustrated in figure 5.5.

The stiffness of unbound materials depends on the level of stress. The greater the stress level of the coarsely granular materials used in pavement layers, the greater their stiffness modulus. The total stress level at different depths of the structure is always calculated separately for each structure under study. If the layer thickness changes during the design process, the modulus of the structural layer material is corrected to correspond to the stress level according to the stress dependence characteristics of each material. The stress level dependence of all the materials used in the design of structural layers must be known.

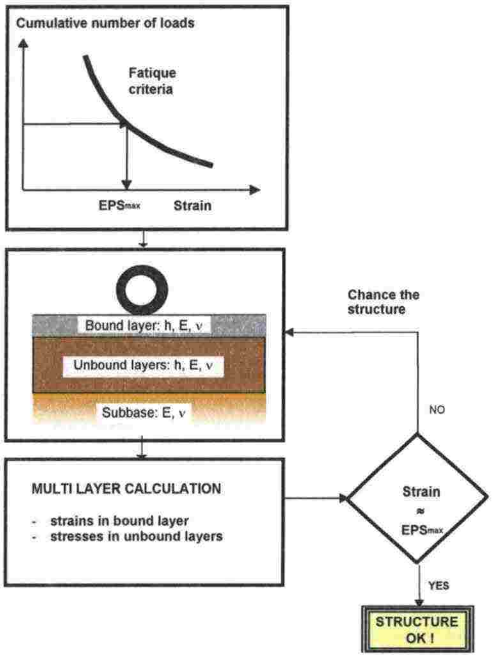

Figure 5.5. Fatigue design process. /14/

The figure 5.6 shows the structural layer dependence between the total stress level and the stiffness modulus of certain "type materials" used in structural layers.

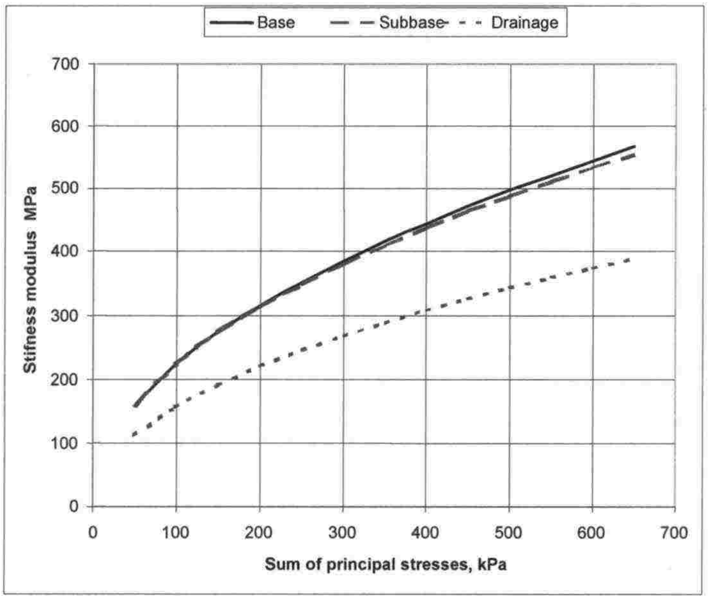

Figure 5.6. Relation between the total stress level ($\theta = \sigma_1 + \sigma_2 + \sigma_3$) and stiffness modulus of certain unbound materials. /16/

5.6 Design of thin pavement (< 80 mm) roads

Design procedure and level of damage

This design procedure is intended for conventional lightly paved roads, which are thin paved AC roads (bound layer thickness before improvement 40...80 mm) and SAC (soft asphalt concrete) paved roads. This design procedure is applicable on roads with a maximum ADT of 3000 vehicles/day.

This design procedure deals with structural improvement. If the grade line is lowered, mass is replaced or road direction is changed, the design procedure for new roads is applied.

This design procedure strives to limit the formation of cracks and longitudinal and cross-directional unevenness caused by frost. Cracks caused by frost are mostly longitudinal cracks and their formation is predicted on the basis of frost lifting. Cross-directional unevenness is described by the height of the crest between ruts. (Figure 5.7.).

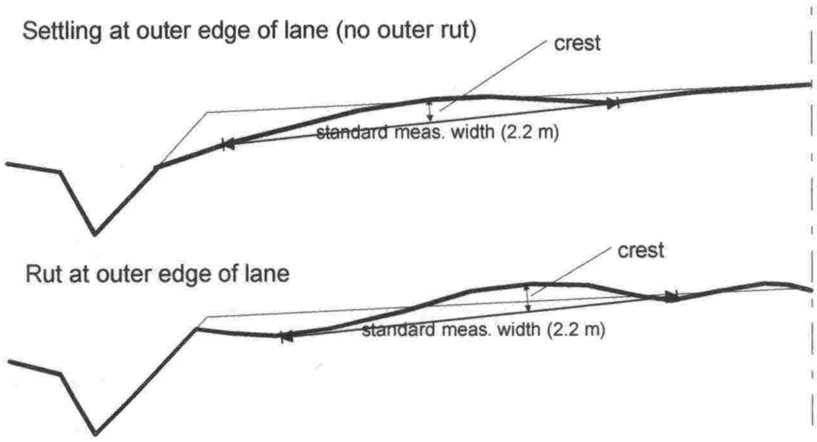

Figure 5.7. Principle of determining crest height.

Design criteria

Maximum allowed frost heave on main roads is 50 mm and on local roads, 100 mm. The necessity of measuring frost heave in improvement projects is determined on the basis of action thresholds (table 5.2).

Table 5.2. Action thresholds of longitudinal cracks.

Crack width mm	**Crack quantity** m/100 m
5-20	≥ 80 m/100 m
20-30	≥ 20 m/100 m
>30	not dependent on crack qty

The values in table 5.3 are used as design criteria for longitudinal unevenness and crest height. Instead of absolute values it is possible to use the rate of increase in IRI and crest height.

Table 5.3. Design criteria of longitudinal unevenness and crest height.

Speed limit km/h	**IRI** mm/m	**Crest** mm
100	2.4	17
80	3.0	21
50-60	3.7	24

The design of structural improvement is completed in phases.

Cracks caused by frost

The quantity of longitudinal cracks in the old pavement is compared to the action threshold. If the action threshold is exceeded, the total thickness of frost-resistant layers is specified so frost heaving of the structure at freezing temperatures equivalent to the design winter does not exceed the allowed amount of frost heave (design criterion). The necessary frost coefficient of the substructure is specified by back-calculating from the measured frost heave of the old structure.

Longitudinal unevenness

The rate of increase in IRI of an old and improved pavement is specified by calculating from the subsoil material type (clay/other), frost heave during the design winter and the thickness of the structure. The calculated results are calibrated with the measured IRI of the old structure.

If the IRI exceeds the maximum allowed value during the design period, the total thickness of the frost-resistant layers is increased to keep the IRI within the allowed limits during the entire design period.

Cross-directional unevenness

Cross-directional unevenness is described by the height of the crest between ruts. The rate of increase in crest height is estimated on the basis of permanent deformation in the upper unbound layer, i.e., the load-bearing layer (Figure 5.8.). Specifying permanent deformation in the load-bearing layer is based on vertical elastic compression (ε_e), which in turn is estimated on the basis of FWD flexing moduli and structural information.

Growth in the crest height of an improved structure is calibrated using the ratio between measured and calculated crest height of the old structure. In selecting an improvement procedure, the total thickness of the frost-resistant layers specified in the preceding paragraphs is taken into consideration. Other influencing factors are pavement quality and thickness, upper unbound layer quality and thickness, road (pavement) width, slope inclination and frost heaving.

To specify the rate of growth of crest height, information is needed on the old structure, its condition, traffic and climate (Figure 5.8).

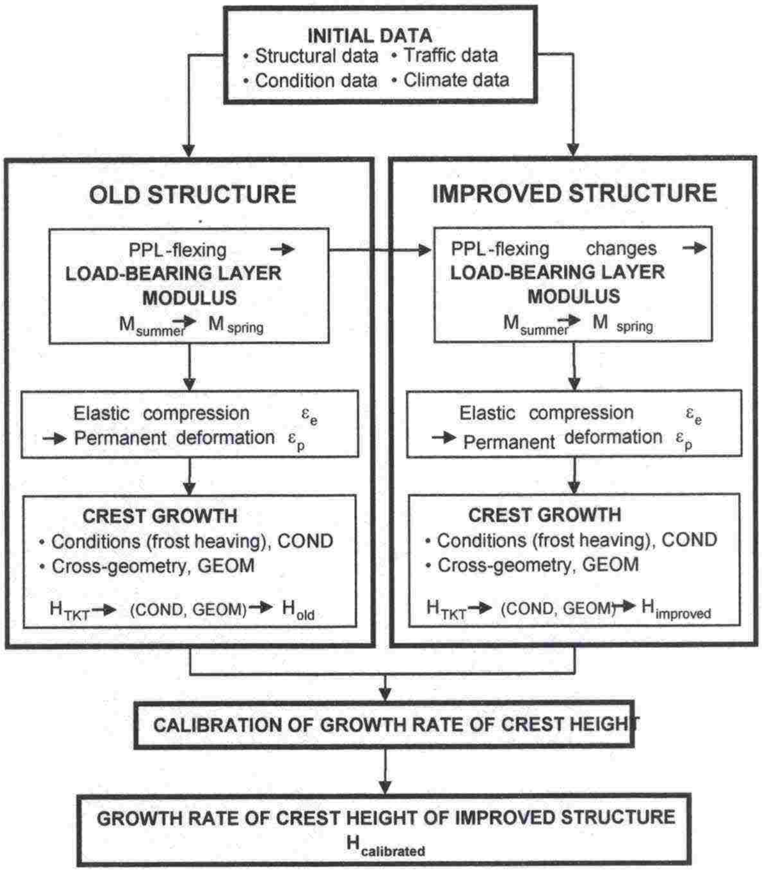

Figure 5.8. Principle of designing the rate of growth in crest height.

Relative permanent deformation of the load-bearing layer is specified on the basis of elastic compression using equation 5.2. The parameters a' and b in the equation are specified on the basis of studies conducted with a road structure study apparatus (TKT-equipment).

$$\varepsilon_p = a' \cdot \varepsilon_e \cdot N^b \tag{5.2}$$

where,

ε_p = cumulative relative permanent deformation of the load-bearing layer [μm/m/a]

ε_e = relative elastic compression of the load-bearing layer [μm/m]

a' = constant (0.06)

N = number of loadings

b = 0.6 in summer conditions

in spring conditions the function for fine material content HA #0.063 in the load-bearing layer:

HA < 4 %	⇒	b = 0.6
4 % ≤ HA ≤ 10 %	⇒	b = 0.4 + 0.05HA
HA > 10 %	⇒	b = 0.9

The calculated crest growth per year is specified with the help of relative deformation using equation 5.3.

$$H_{TKT} = 0.3 \cdot (\varepsilon_{p\,spring} + \varepsilon_{p\,summer}) \tag{5.3}$$

where,

H_{TKT} = crest growth per year on the basis of TKT tests [μm/a]

coefficient 0.3 = default value of thickness of load-bearing layer [m]

Conditions (frost heaving) and cross-geometry are taken into consideration with the help of the COND and GEOM coefficients (equation 5.4).

$$H_{\mathrm{old/improved}} = H_{TKT} \cdot COND \cdot GEOM \tag{5.4}$$

where,

$H_{old/improved}$ = calculated crest growth per year while taking geometry and condition factors into consideration [μm/a]

COND = condition coefficient

GEOM = cross-geometry coefficient

The rate of growth of the improved crest is calibrated with the ratio of the calculated and measured rate of growth of the old structure using equation 5.5.

$$H_{calibrated} = \frac{H_{measured}}{H_{old}} \cdot H_{improved} \qquad (5.5)$$

where,

$H_{calibrated}$ = calibrated rate of growth of crest height [mm/a]

$H_{measured}$ = actual measured annual rate of growth of crest height [mm/a]

5.7 Bitumen stabilization

Bitumen stabilization refers to a station or site mixing method in which the bearing capacity of a road's base or sub base layer is increased by binding cold aggregate material with either foamed bitumen or a bitumen emulsion.

Stabilized masses differ so much from hot mix asphalt masses that stabilized masses should be tested using test procedures adapted just to them. For this reason design parameters should be specified using triaxial testing. Mix design of masses should also take the stabilization technique into consideration, and materials suited to actual conditions should be used in the laboratory masses /9/.

Based on study results, both foam and emulsion bitumen can be used as a binder in bitumen stabilization. Bitumens from B160/220 to B650/900 are suitable as a base bitumen in both cases. Even extra hard bitumen (B20/30) can be used if it is emulsified. From the standpoint of workability, stabilization is most successful when soft bitumens are used /9/.

As a pavement material bitumen stabilization is situated somewhere between unbound and bound materials. Its damage mechanism does not involve fatigue in the traditional sense. Damage is largely based on strain or stress exerted vertically on the layer.

The clearest observation with all the test structures over several years was a decrease in stress from loading when measured under the stabilized layer. This is the result when water slowly escapes from the stabilizing layer and makes possible a post-compaction of the structure. The same phenomen could not always be detected with the help of bearing capacity calculated on the basis of FWD measurements.

Follow-up measurements indicate that the rate at which the structure become stiffer is the quickest immediately after stabilization, as is apparent in figure 5.9. Small differences were also noticed between different binding agents.

The results indicate that special attention should be paid to the design of bitumen stabilization. Incorrect mass or an inadequate structure easily leads to insufficient initial stiffness under loading and rapidly developing damage. Composite stabilization is worth considered at sites requiring high initial stiffness. The wearing course on top of the stabilized layer should be constructed immediately after stabilization, and the load distributing characteristics of the course should be taken into consideration.

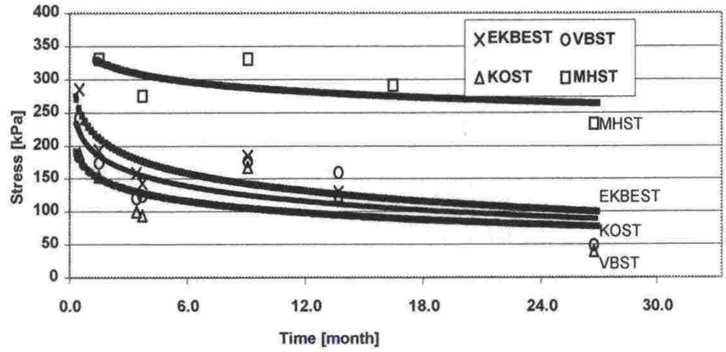

Figure 5.9. Development of stresses caused by truck axles during the follow-up period at the Nakkila test site. (EKBEST= emulsion stabilisation with extra hard bitumen, VBST= foam bitumen stabilisation, KOST= composite stabilisation, MHST= stabilisation with granulated blast-furnage slag)

Bitumen stabilization usually decreases the original amount of damage at a rehabilitation site for several years. Stabilization does not, however, eliminate damage caused by frost heave in the spring, but when the mass is correctly mix and thickness designed, the structure appears to recover nearly undamaged at least in terms of evenness. Based on monitoring results, it is not recommendable to assess stabilization solely on the basis of bearing capacity results obtained with a FWD.

5.8 Composite structures

A bitumen-cement composite is a base or sub base layer of a road's pavement containing both bitumen and cement as a binding agent. A composite exploits the basic characteristics of both binding agents in the same mass; both sufficient rigidity and flexibility. The goal of the composite structure studies were to determine the structural and functional principles, design parameters and main design parameters of the composite material model.

Preliminary laboratory tests were used to study five different composites. The following three most promising were selected for further studies:

- open asphalt concrete grouted with cement mortar
- a composite of recycled asphalt and cement mortar
- composite stabilization.

The chosen composites were studied in test structures from 1996 to 2001 /1...7, 35/.

Open asphalt grouted with cement mortar functioned well in laboratory tests and in a test road. It was the most asphalt-like of the tested composites. With currently used construction methods this type of mass is only suitable for small-scale special applications, like storage areas, where heavy static loads are used.

Recycled asphalt and cement mortar composite was determined in the studies to be a very promising material. The possibility to recycle the asphalt and the usability of the bitumen they contain make the mass a competitive product in terms of life cycle cost. It combines fatigue resistance that is better under minor and greater strains than that of the other studied composites, relatively high stiffness and good frost resistance /12/.

Composite stabilization is perhaps the most interesting of the studied composites due to its simple construction technique and abundant possibilities of modification. With the combined effect of two different kinds of binding agents it is possible to significantly influence the stiffness of a structure and adjust it to the desired level.

Composite stabilization (marked Comp G in fig 5.10) with the composition used in the laboratory test phase (3 % cement, 3 % bitumen) was rigid, but relatively brittle. It was the most concrete-like of the studied composites. With a high enough cement content its frost resistance is good according to the frost resistance criteria of concrete structures. On the other hand, a lower cement content avoids excess rigidity on weak subgrades. A definite benefit of composite stabilization compared to bitumen stabilization is its faster development of initial strength.

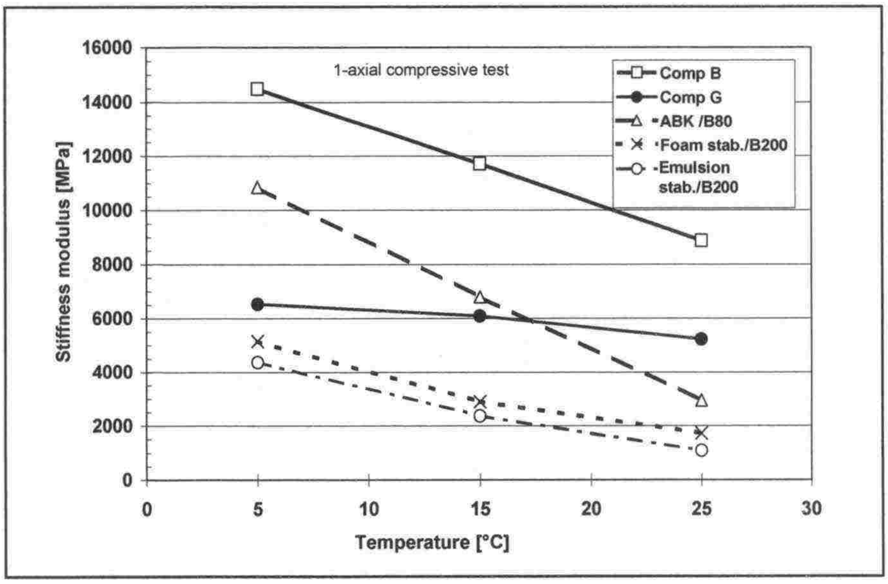

Figure 5.10. Stiffness modulus M_{rk} of asphalt chip and cement mortar composite (B), composite stabilization (G) and emulsion and foam bitumen stabilization (uni-axial compression test). ABK = AC in base course /12/

The fatigue curves of all the studied composites are quite low compared to asphalt, for example, due to the high stiffness of the composites Figure 5.3. Especially the small slope of the fatigue curve of composite stabilization, which has a high cement content, indicates that its fatigue is nearly independent of the amount of strain. At the same time, however, the allowed strain of composite stabilization should be low regardless of the structure.

5.9 Fatigue resistant asphalt structure

Thick hot mix asphalt structures were used to study how the functional characteristics of different kinds of asphalt materials could be utilized as efficiently as possible in the different parts of a pavement. The most interesting of the tested structures was a fatigue resistant structure in which suitable materials were used to prevent wear and deformation as shown in figure 5.11. The bottommost bound layer is the structure's most important layer from the standpoint of durability, and therefore special attention is given to its design.

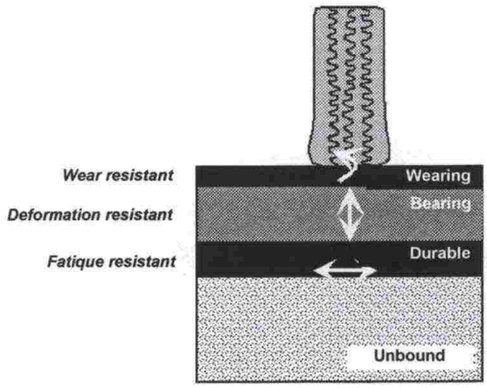

Figure 5.11. Principle of a fatigue-resistant structure

A fatigue resistant test structure, AC (B-200) + ACB (Gilsonite 17 % of the binder), was compared to the normal structure of a high volume road, ACB + SMA, on Ring II /10, 35/. The bottommost bound layer of the test structure was an AC layer that withstands tensile stress well but is not rigid. Laboratory tests indicated that its ability to withstand tensile stress is 100 times greater than that of conventional ACB. The next layer up was very stiff ACB, which effectively distributes a load. The tensile resistance of ACB was not important, because the layer was near the neutral bending axis, and was not subject to tensile stress. The wear layer in both structures was normal SMA, which was planned to be made two years after the road was opened for traffic (figure 5.12).

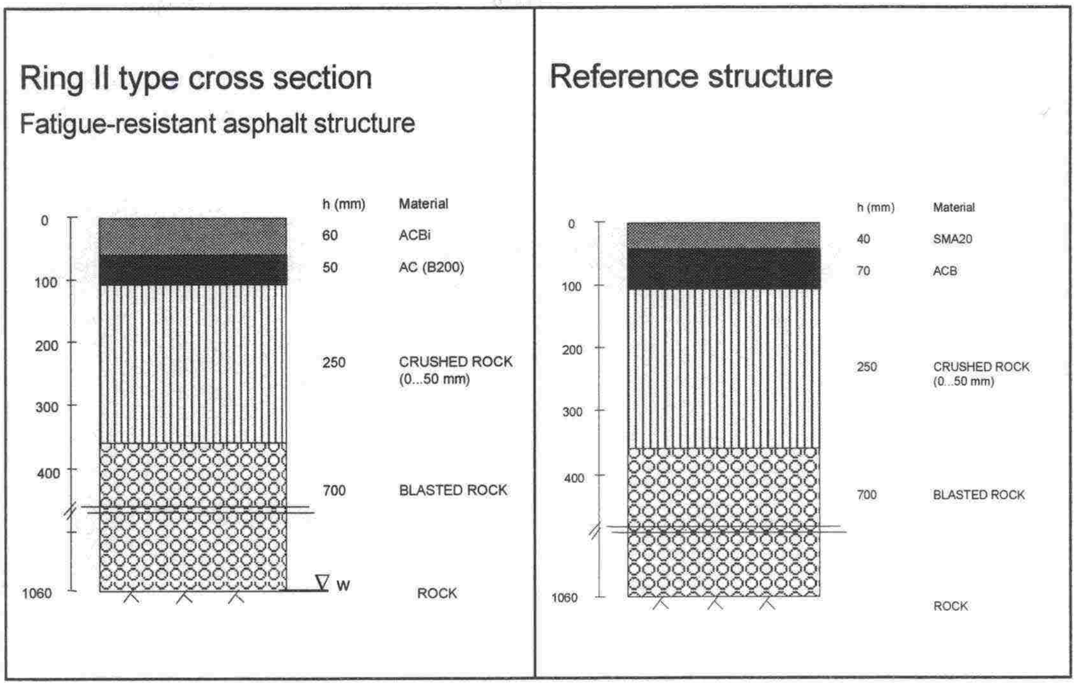

Figure 5.12. Ring II test structure and reference structure /10/

Figure 5.13 presents the stiffness modulus of the materials at different temperatures. It can be seen from the figure that the relative stiffness of ACB is emphasized, especially at high temperatures. The fatigue curves of the bottommost bound layers are presented in figure 5.14, where it can be seen that according to laboratory tests, AC (B-200) withstands 100 times more loading at the same strain level than ordinary ACB.

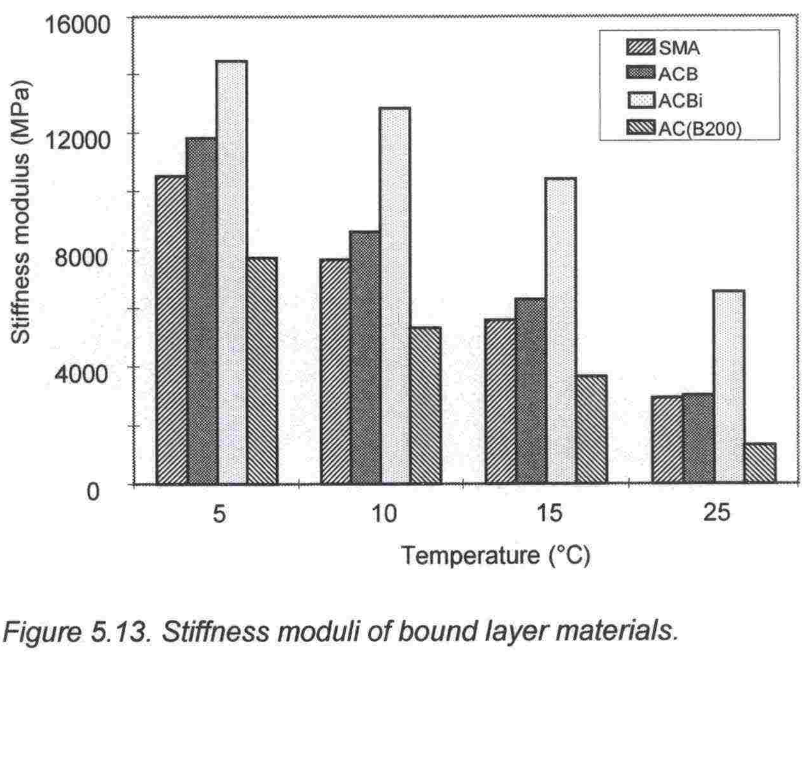

Figure 5.13. Stiffness moduli of bound layer materials.

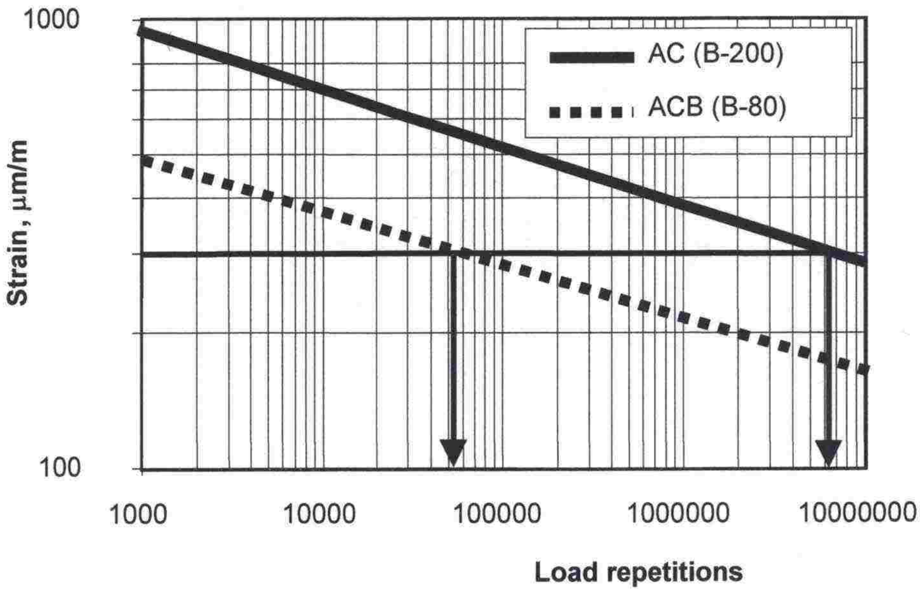

Figure 5.14.Fatigue curves of the bottommost bound layers.

Strain gages were installed in the pavement of test structures. Gages were used in the top and bottom layers of the pavement in the longitudinal and transversal directions with respect to the loading direction. Also, pressure sensors were installed in the base course and deflection sensors were installed in the surface of the pavement.

The test structures were loaded with an HVS-Nordic test machine. At the beginning of the testing both structures were subjected to a large number of response tests using different tyre types, tyre weights and tyre pressures. During the actual test the loading wheel was a 60 kN dual wheel. The experimental structure was loaded 500,000 times and the conventional structure was loaded 140,000 times.

Naturally, no cracks appeared in either structure due to the low number of loadings, and rut formation consisted of a few millimeters of initial compaction. The functionality of the structures was compared on the basis of response measurements and calculations.

The relative age of the test structure was calculated as 30 times that of a conventional structure. On the other hand, the calculations indicated that an 80 mm thick test structure withstands as much as a 110 mm conventional structure.

Life cycle cost calculations were made for the structures for a 40-year period according to a procedure developed in the TPPT-project. APAS thickness design program parameters were used as fatigue criteria, which were significantly more on the safe side than the laboratory results. Also, when calculated this way the annual costs of the test structure were 6-7 % lower than those of a conventional structure.

Based on the results of the test it can be concluded that for approximately the same price the test structure provides significantly more traffic capacity than a conventional structure. Thus, it is possible to use several maintenance strategies, as the bottommost asphalt layers remain intact and allow milling and resurfacing in addition to just adding a new slab.

A FWD measurements were conducted when the road was completed in October 2000. The temperature of the pavement was 12 °C. The bearing capacity of the section with a conventional structure varied from 488 to 838 MPa with an average value of 651 MPa. The bearing capacity of the test section of road varied between 530 and 903 MPa, with an average value of 730 MPa. The difference in load-bearing capacity was due to differences in the thickness of the bound layers, contrary to plans.

In the response measurements a single wheel caused about 100 % greater stress in the base course and strain at the bottom of the pavement in the conventional structure than in the test structure.

According to laboratory tests the test structure would withstand 100 % more loading even with the same strain, so the test structure can be expected to last many times longer than a conventional structure. After a year of monitoring no damage was detected along the test sections of road.

A fatigue resistant asphalt structure is suitable for a road which has considerable heavy traffic, where the road's damage mechanism consists of fatigue or rut formation in the unbound layers. Because the minimum thickness is high, the advantages of the structure can only be utilized with large traffic volumes. The test site has only been monitored a little over a year, but already it appears to be very suitable for a heavily trafficked road /10/.

5.10 Granulizing moraine by pelletization

The winner of TPPT's sustainable development road structures idea competition arranged in 1995 was a "Round balls" proposal. It proposed refining glacial till by pelletizing it with the help of cement or bitumen /27/.

Finland's most common soil type is glacial till, which covers nearly half of the country's land area in the form of basal till /28/. As a mixed soil type till contains a varying amount of different grain sizes from large rocks to small clay particles, both of which in large amounts weaken the earth construction characteristics (handling characteristics, load-bearing capacity, frost resistance) of till /29/.

The stoniest gravel tills have been used in earth and road structures in crushed form many years already /30/. Use of the fine-grained silt-like sandy tills and silt tills has been limited to embankments in dry conditions, and even then the slopes have to be protected against erosion /29/. The principle of the proposal that won the idea competition was to improve the usability of fine-grained tills by reducing the amount of fine-grained till material by means of pelletization /27/.

Pelletization of till involves coarsening treatment of its grains with a binder so the amount of fine material (grain size < 74 μm) in the till decreases and the grain size distribution becomes coarser as the fine particles adhere to each other and larger grains /31/.

It has been succeeded to pelletize prescreened tills with both cement and bitumen emulsion in laboratory as well as in field tests. Durability tests in laboratory indicated that the cement pellets became pulverized about 20 % and the bitumen pellets 20-50 %, depending on the amount of bitumen and water /32,33/.

Test structure

The test structure with cement pelletized till is located on a pedestrian and bicycle ways next to road section 2 of route 304 between Toijala and Valkeakoski. The length of the test structure is 80 m. The reference structure is 350 m long /32/.

Table 5.4. Materials, assumed design moduli and calculated structural layer thicknesses of the experimental and reference structures.

Structural layer	Material	Design modulus [MPa]	Structural layer thickness [mm]	
			Test structure	Reference structure
Wearing course	Ab 16/100	2500	40	40
Base course	CRA 0/50 mm	200	150	150
Sub-base course	CRA 0/50 mm	200		300
	Till –pellets	150	300	
Filter course	Sand	50	300	300

The base-course of the test structure was constructed out of pellets stored seven weeks in a covered stockpile. The base-course of the reference structure was made out of crushed rock aggregate (CRA) 0/50 mm; otherwise the structures were similar. The grain size distributions of the used aggregates are presented in figure 5.15.

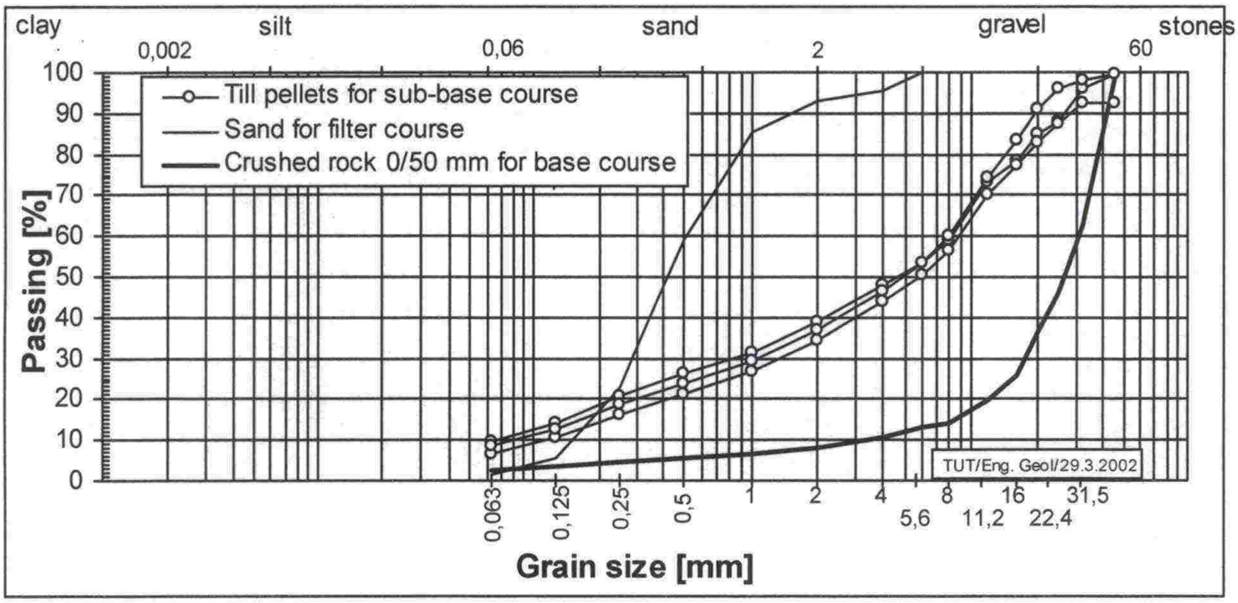

Figure 5.15. Grain size distributions of aggregates used in the till pellet test and reference structures.

Layer thicknesses were designed using the traditional Odemark's equivalence method.

The prescreened till was pelletized at a concrete mixing plant. Cement and necessary additional water were mixed in 0.75 m^3 batches using the station's double-axle mixer and pelletized in 3 m^3 batches in the transport tank of concrete mixer trucks. The till was not coarsened as much and as homogenously as in similar smaller-scale field tests conducted in the autumn of 1997. When loaded and spread, the till pellets acted like unbound aggregate /32/.

The results of FWD-tests made on the test and reference structures are compared in figure 5.16. In the first monitoring measurement on 23.10.1998, one week after asphalting, deflection in the till pellet-structure was already somewhat smaller than in the reference structure. In later measurements the strengthening of the pellet structure can be clearly seen /34/.

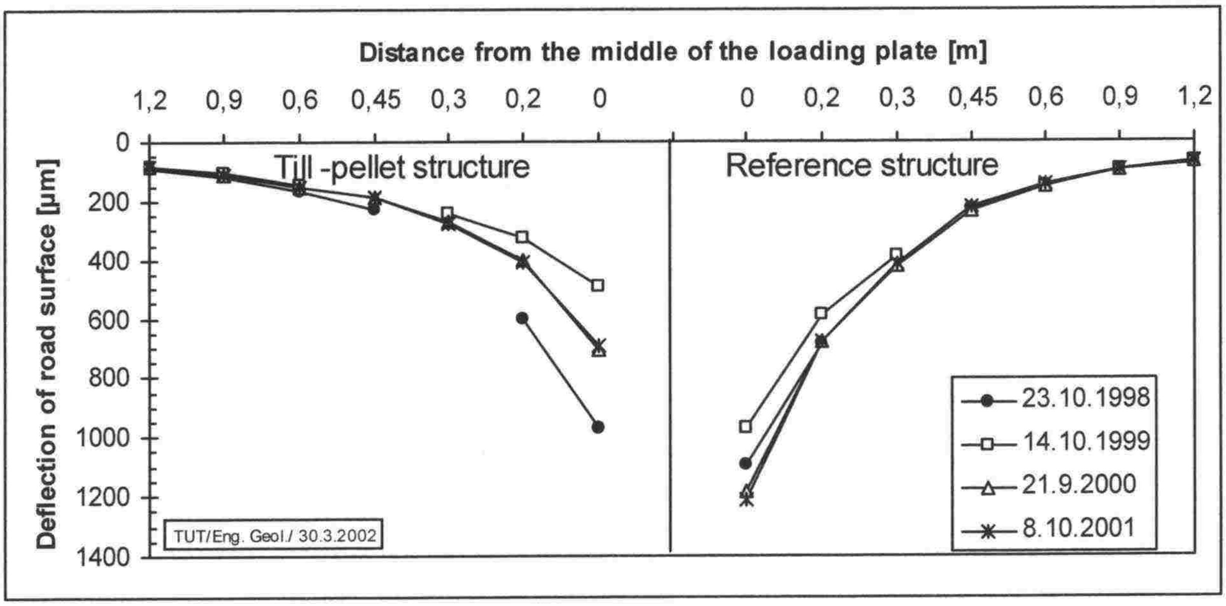

Figure 5.16. Deflection of till pellet experimental and reference structures at different times, measured with a FWD.

The field test of till pelletization indicated that coarsening glacial till with cement is successful not only in the laboratory, but also on a larger scale. Coarsening in the latter case is not necessarily as effective or homogenous as in the laboratory. The till pellet layer in the completed structure has begun to act like a bound layer. During the last two monitoring years the E2 value of the test structure has been about 70 % higher than that of the reference section /34/.

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6 HVS-NORDIC-THE HEAVY VEHICLE SIMULATOR

6.1 HVS-NORDIC - accelerated pavement testing facility

The accelerated pavement testing (APT) facility can be used to test pavements under full-scale adjustable conditions. APT provides information about the durability of a road structure in just a few months, whereas the process would take up to twenty years with ordinary test roads loaded by normal traffic. As a procedure, APT is a link between laboratory studies of a pavement structure and test roads.

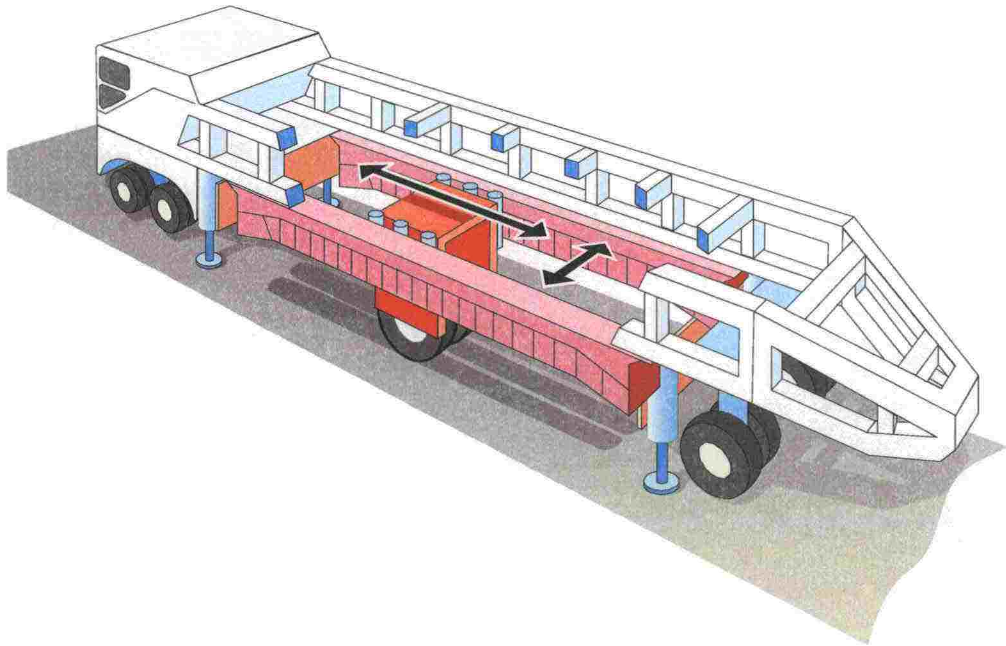

Figure 6.1. HVS-NORDIC accelerated pavement testing facility (Ingemar Franzén, Ny Teknik)

6.1.1 HVS-NORDIC's features

The dimensions of the machine are: length 23 m, width 3.7 m and height 4.2 m, and it weighs 46 tons. The length of the loaded area is 8 m and the width is adjustable up to 1.5 m. The loading wheel can be a conventional dual wheel or a single wheel (wide base tyre). The wheel load can be 20 - 110 kN, corresponding to an axle load of 4 – 22 tons. The speed of the loading wheel can be selected 1 - 12 km/h. The wheel load can be supplemented with an additional dynamic load to simulate the additional stress caused by unevenness of a road.

A significant feature of the HVS-NORDIC facility compared to many other types of APT facilities is that the direction of travel of the loading wheel is in the "direction of the road line". The loading wheel can be programmed to move in the lateral direction over an loaded area up to 1.5 m wide. Loading can take place in one direction or both directions, whereupon the number of load repetitions is doubled.

An essential part of the machine is a heating/cooling unit with which the pavement being tested can be kept at a desired temperature. This is necessary so that all the test structures can be subjected to comparable conditions in the climate of the Nordic countries. This feature is included in a mobile APT facility the first time in the world in the HVS-NORDIC.

HVS-NORDIC can be used continuously, 24 hours a day and 7 days a week. The maximum speed of the continuous load is 12 km/h. The maximum number of load repetitions with bi-directional loading mode is 25,000 a day and 750,000 a month.

In conjunction with the purchase of the HVS-NORDIC , two test basins were constructed at Otaniemi, in which it is possible to construct full-depth road structures. The basins are 3 m wide at the bottom and 4 m wide at the surface, 32 m long (+ 16 m slope) and 2.5 m deep (figure 6.2). The water level in the basins can be regulated to change the moisture conditions of the road structure.

Figure 6.2. VTT's test basin at Otaniemi in Espoo.

6.1.2 Finnish-Swedish co-operation

The APT facility HVS-NORDIC was purchased in cooperation with the Swedes. The participants in both countries are national road research institutes and road administrations. A six-year common research project lasting from 1997 to 2003 was set up with the Swedes to serve the needs of both countries. A mutual research program was compiled to ensure that the research benefits both countries. The program includes quality control, instrumentation, measurements, observations, load parameters, analysis and reporting related to experimental structures. All the construction and measurement data of the tests are stored in a common database for later use in analysis done by the participants.

6.1.3 Use of APT in pavement design

Tests performed with the APT facilities play a key role in acquiring research data and information about the pavement performance. This information is needed when modeling the behavior of a road structure under moving wheel load and when evaluating the lifetime of a road. The APT facility can be used to test pavements constructed with new materials. The APT facility is especially well suited to comparing different types of pavement structures.

The APT facility makes it possible to determine the impact of traffic load on a road surface and pavement and to find out the deterioration mechanism of a road structure. But, because the test is accelerated, the long-term effect of climate factors has to be evaluated in another way. Nevertheless, by changing the test conditions it is possible to obtain indicative information on the impact of weather conditions on the deterioration process of a road.

6.2 Test case: Thawing structure

6.2.1 Objective of the tests

The objective of the HVS test was to study the behavior of a road with a frost-susceptible subsoil and light pavement under a traffic load during the thawing phase /1, 2/. Another goal was to compare the performance of an conventional pavement and a reinforced pavement.

6.2.2 Test structures

Three identical pavements were constructed in the test basin at Otaniemi, where it was possible to regulate the ground water level. The subsoil was frost-susceptible dry-crust clay. A separating, load-bearing layer of unbound materials and a wearing course of asphalt concrete were constructed on top of the clay as shown in figure 6.3. One test road section was reinforced with a steel mesh installed in the load-bearing layer.

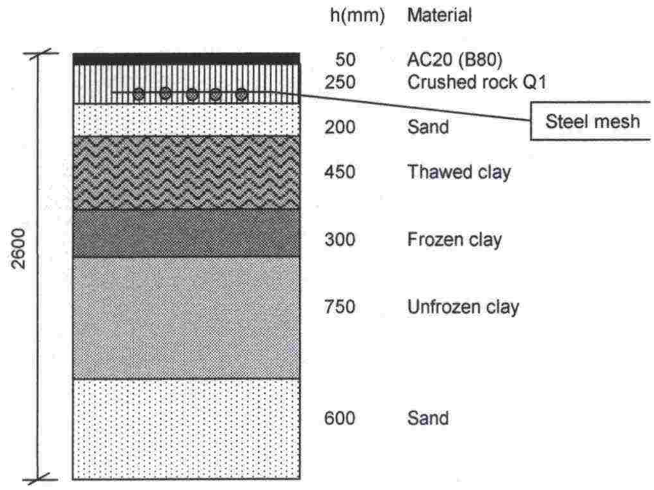

Figure 6.3. Reinforced test structure.

Tensile sensors were installed in the test structures in the top and bottom surfaces of the pavement in the longitudinal and cross directions with respect to the load direction. Pressure sensors were installed in the different layers and a deflection sensor was installed in the surface of the pavement. Moisture, pore pressure and heat flow sensors were installed in the layer of clay. To measure the vertical temperature profile of the entire structure, thermocouples were installed in the different layers.

6.2.3 Test program

The test structures were allowed to freeze naturally during the winter. Because of the mild winter, the structures were artificially cooled from the surface in late winter to obtain a frost depth of 1.4 m. After freezing the frost was allowed to thaw so that 0.5 m of the surface of the layer of clay was thawed. The test structures were loaded at this phase of the thawing process.

At the beginning of the test, response measurements were made using different load parameter values. The first test employed a 50 kN load with dual wheels and the last two tests employed a 40 kN load with dual wheels. The loads in all the tests were applied in one direction.

6.2.4 Test results

The first structure was loaded with a wheel load of 50 kN, and the structure broke as a result of a 100 mm rut formed after a few thousand passes. For this reason the second and third structures were loaded with a smaller wheel load. Figure 6.4 presents the rut formation as the result of these tests. The reinforced structure has clearly less rut formation than the unreinforced structure. When loaded to the same rut depth, the reinforced structure withstood 150 % more loading than the conventional structure.

Less cracking also appeared in the surface of the reinforced structure, or an equivalent amount of cracking required more loading (figure 6.5).

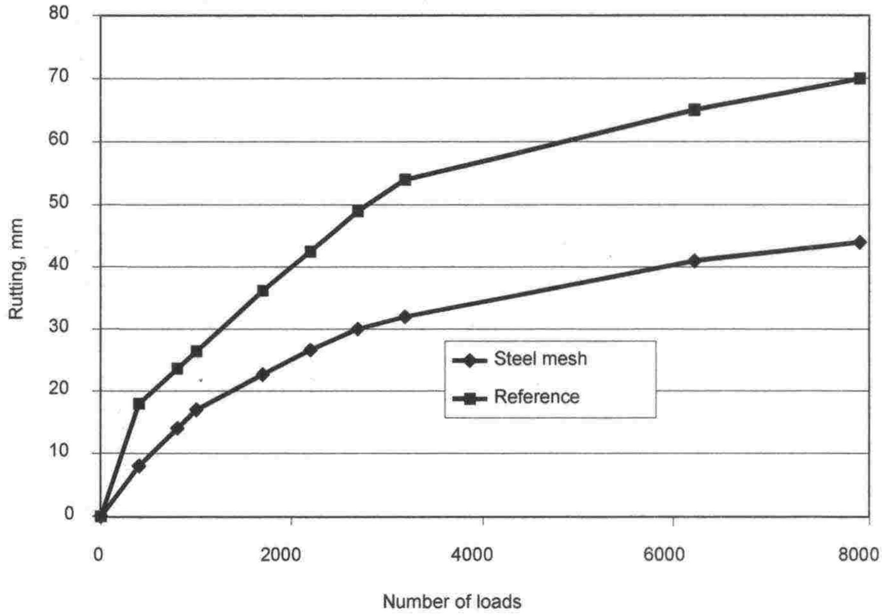

Figure 6.4. Rut formation as a function of load repetitions.

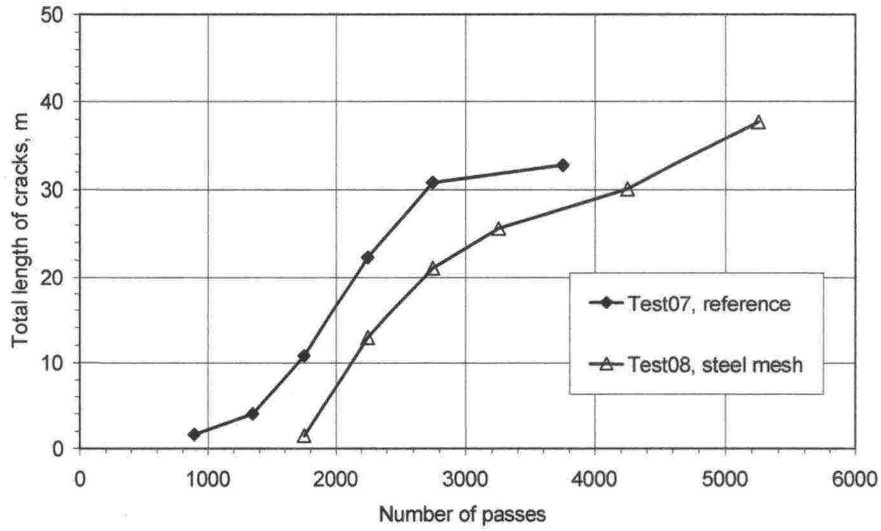

Figure 6.5. Crack formation as a function of load repetitions.

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6.3 Test case: Sloped Structure

6.3.1 Objectives of the tests

The HVS tests /1/ were part of the thinly pavement roads - project. The objective of the tests was to study the impact of a road's cross section, particularly the location of the slope and load, on the permanent deformations of the pavement, i.e., rut formation in the structural layers. The purpose of the test results was to verify the design procedures and calculation models developed in the project.

6.3.2 Test structures

There were three tested structures: one without slopes, one with gradual slopes (1:3) and one with steep slopes (1:1.5). The tested road structures were designed to correspond to the pavement of the low-volume road network. The total thickness of the pavement was 650 mm. Underneath a thin AC pavement (50 mm) was a 400 mm thick load-bearing layer of crushed rock. Under that was a 200 mm layer of gravel and the subsoil was dry-crust clay. The slope extended to the surface of the layer of clay (figure 6.6).

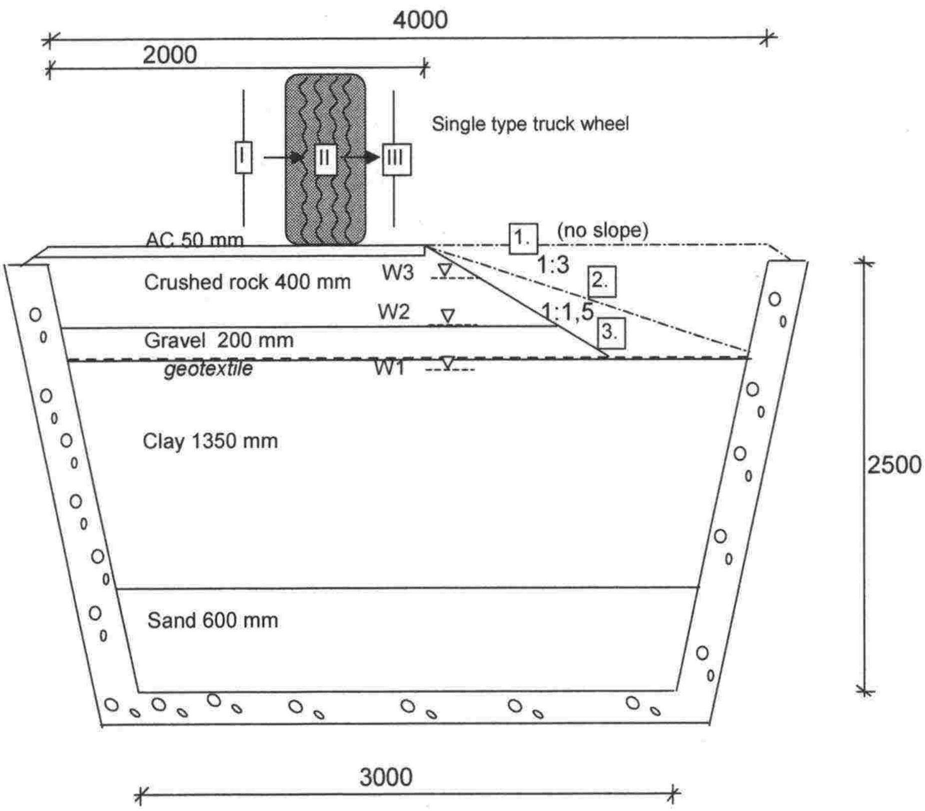

Figure 6.6. Tested sloped structures.

Instrumentation was installed in the test structures so that by monitoring the measurement results it was possible to observe permanent and dynamic deformation of different structural layers in the horizontal and vertical directions. The earth pressure, pore water pressure, water content and bearing capacity of the layers were also monitored. Samples were taken of the structural layers. They were used as test samples for determining the deformation characteristics by means of triaxial tests.

6.3.3 Test program

A super single wheel was used as a load wheel, and the load was increased by 10 kN steps: 30, 40 and 50 kN. The location of the wheel was varied during each loading step so that the load was located 550, 850 and 1050 mm from the edge of the slope. The ground water level at the beginning of the test was in the layer of dry-crust clay about 50 mm below the layer of gravel (W1). As the test progressed to the 50 kN step the water level was raised to the bottom of the layer of crushed aggregate (W2) and even 200 mm higher in the structures with slopes (W3).

6.3.4 Test results

Distribution of permanent deformation in the layers.

The distribution of the permanent deformations in structural layers was defined to each 200 mm thich layer. The pavement was assumed to be non-compressible. The amount of 76 - 83 % of the permanent deformation occurred in the 400 mm base course, and the deformation was approximately evenly distributed between the top and bottom part of the base course. The contribution of the layer of gravel to rut formation in the surface of the structure was 10 - 13 % and that of the top parts of the subsoil (400 mm) was 5 - 6 %. Permanent deformation that occurred in the structures was significant, and it can be assumed that the loading situation in the steeply sloped structure was close to the failure. The structural damages of each structure were easily detectable during the test.

Figure 6.7 presents the relative deformation in the vertical direction in the case of the 1:1.5 slope in the 200 mm layers with different loads and water levels. A significant amount of the deformation is concentrated at the top and bottom parts of the base course. The sliding surface develops to the area of greatest deformations and continues towards the slope. In the steeply sloped structure the greatest lateral deformation was detected in the middle of the slope. In the gradual sloped structure the greatest lateral deformation was detected at the surface of the structure.

Relationship between permanent deformation and dynamic deformation.

The relationship between the development of permanent deformation and dynamic deformation (momentary deformation caused by the load pulse of the wheel) was examined in both horizontal and vertical shifting (figure 6.8). Permanent deformation remains at a reasonable level up to a certain threshold, after which it increased significantly. This threshold value can be compared to the yield strength of soil. When it is exceeded, permanent deformation increases. Structures should be designed so that this threshold value is not exceeded.

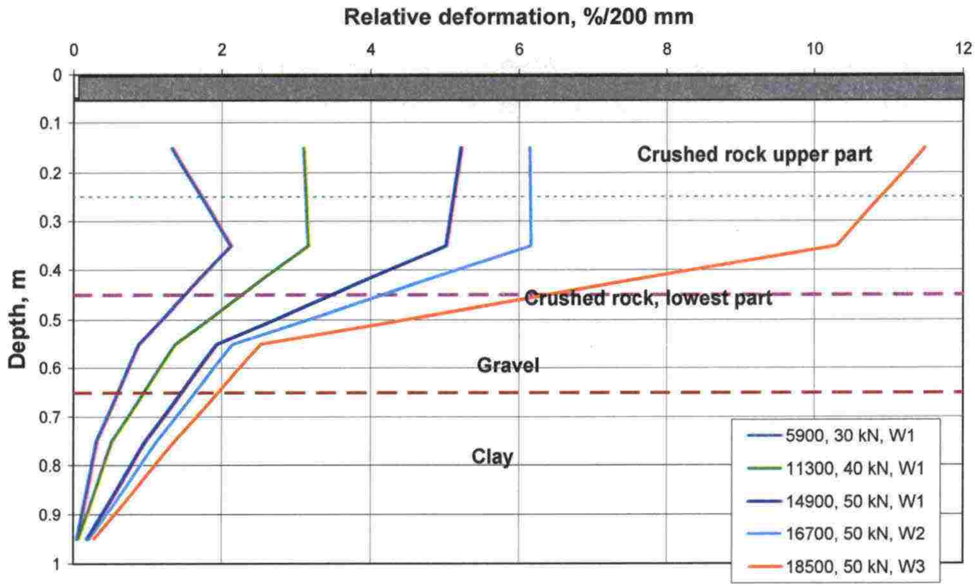

Figure 6.7. Development of relative deformation in different structural layers with different loads and water levels (slope: 1:1.5)

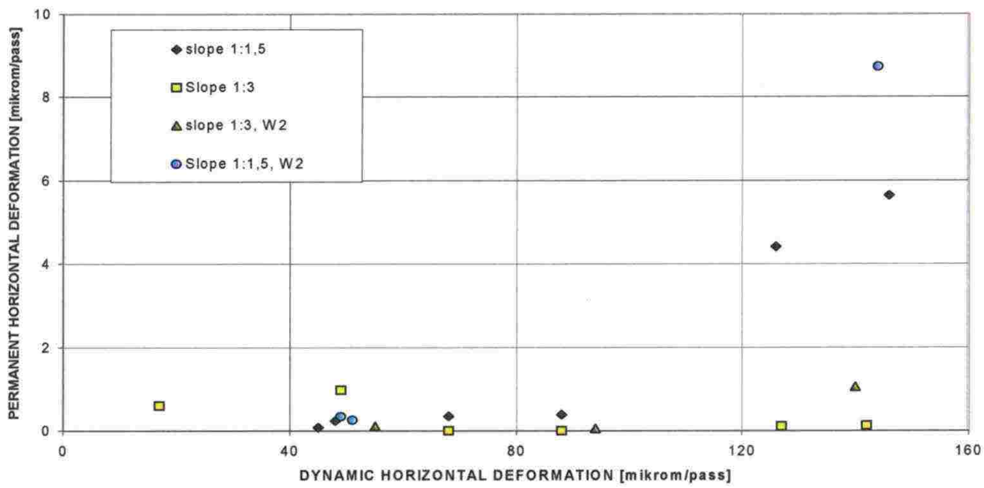

Figure 6.8. Relationship between dynamic and permanent horizontal shifting in the bottom part of the layer of crushed rock with different slopes.

<u>Geometric coefficient</u>

The geometric coefficient, which is used to depict the grow-rate of height of the ridge when designing low-volume roads with thin pavement (chapter 5.6 and /2/), was defined on the basis of the measured results of the HVS tests. With this coefficient it is possible to estimate the impact of different slope inclinations and road widths on rutting (or the height of the ridge) in the pavement. The geometric coefficient (GEOM) is defined using equation (6.1), but so that its value is at least 0.5 (figure 6.9).

$$GEOM = 0.407 + \frac{-0.273 \cdot B^2 + 2.785 \cdot B - 4.971}{2.7^{\left(kalt/3\right)}} \geq 0.5 \quad (6.1)$$

B road width, m
kalt slope inclination, 1:inclination

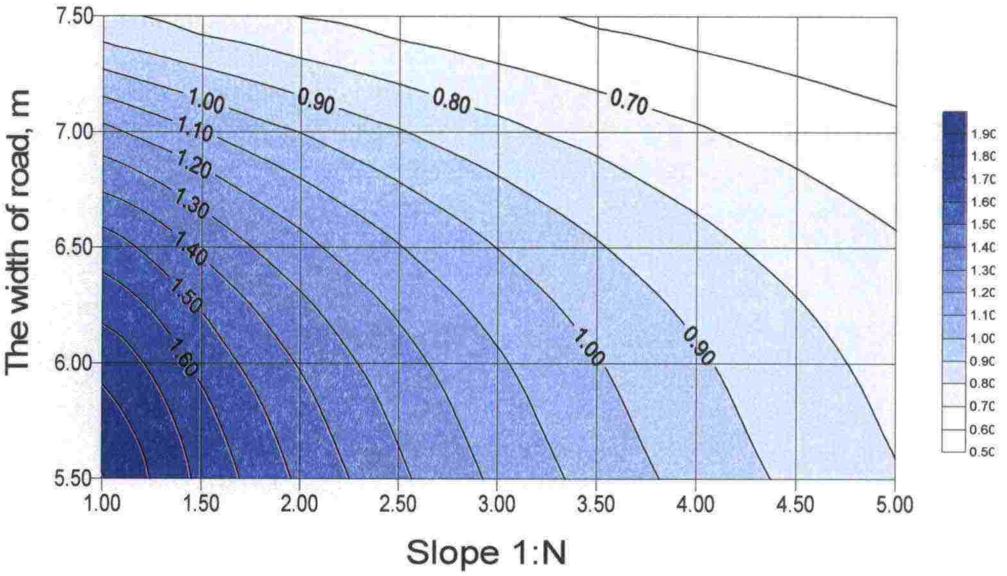

Figure 6.9. Geometric coefficient.

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7 OTHER PROJECTS

7.1 Settlement Calculation Competition

The Finnish Road Administration arranged an international settlement calculation competition in 1997-1999, where the task was to calculate the settlement, horizontal displacements and pore water pressure in the subsoil caused by an test embankment constructed on a 23 m thick, slightly overconsolidated clay. In August 1997 a 3 m high and 100 m long test embankment was constructed in Haarajoki (figure 7.1). Half of the test embankment was built on natural subsoil and half on subsoil containing vertical strip drains (c/c=1.00 m). The purpose of the competition was to determine and improve the accuracy of settlement calculation related to road design and also to determine the methods of expressing the results of the calculations.

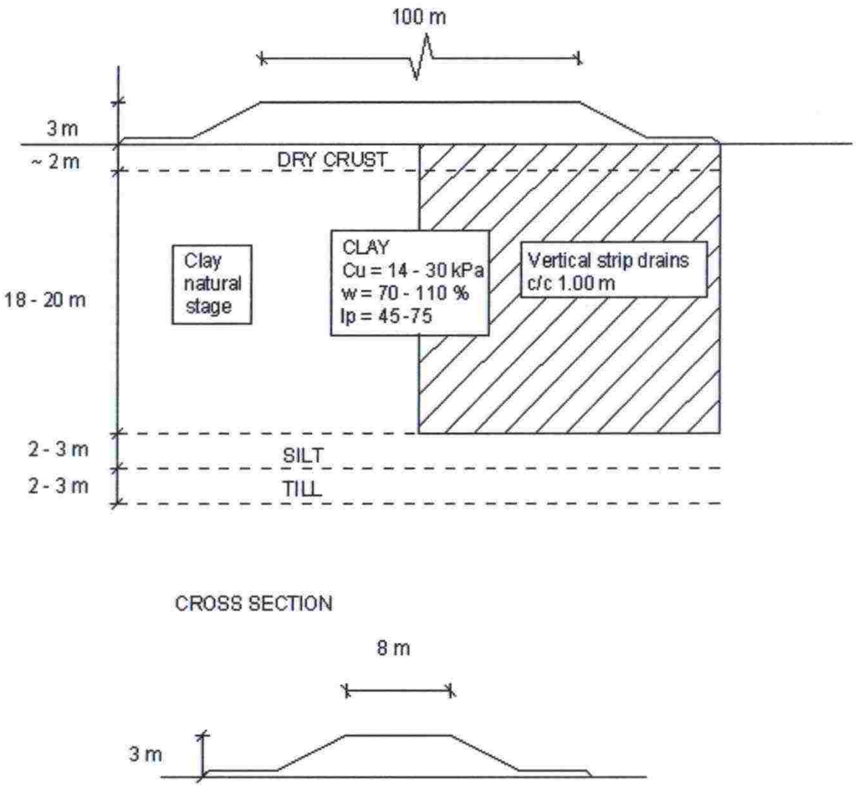

Figure 7.1. Dimensions of the Haarajoki test embankment

The test embankment is well equipped with instrumentation, and settlement, horizontal displacements and pore water pressure has been measured from beginning with the natural state before the embankment was constructed. The embankment is part of a permanent noise barrier and measurements are still being made. The results of the measurements can be viewed on the Internet site at http://www.tiehallinto.fi/tkohj/strprojektit.htm. The results of the competition and information about the embankment were published in a Finnish Road Administration report /1/.

The competition's period of calculation was two years. During this time the embankment constructed on natural subsoil settled 32 cm and the embankment constructed on the subsoil with vertical strip drains settled 53 cm. After four years the respective settlements were 42 cm and 71 cm. The subsoil with vertical strip drains is also still settling rapidly, and based on the observed settlement it can not be concluded that the primary settlement has ended yet.

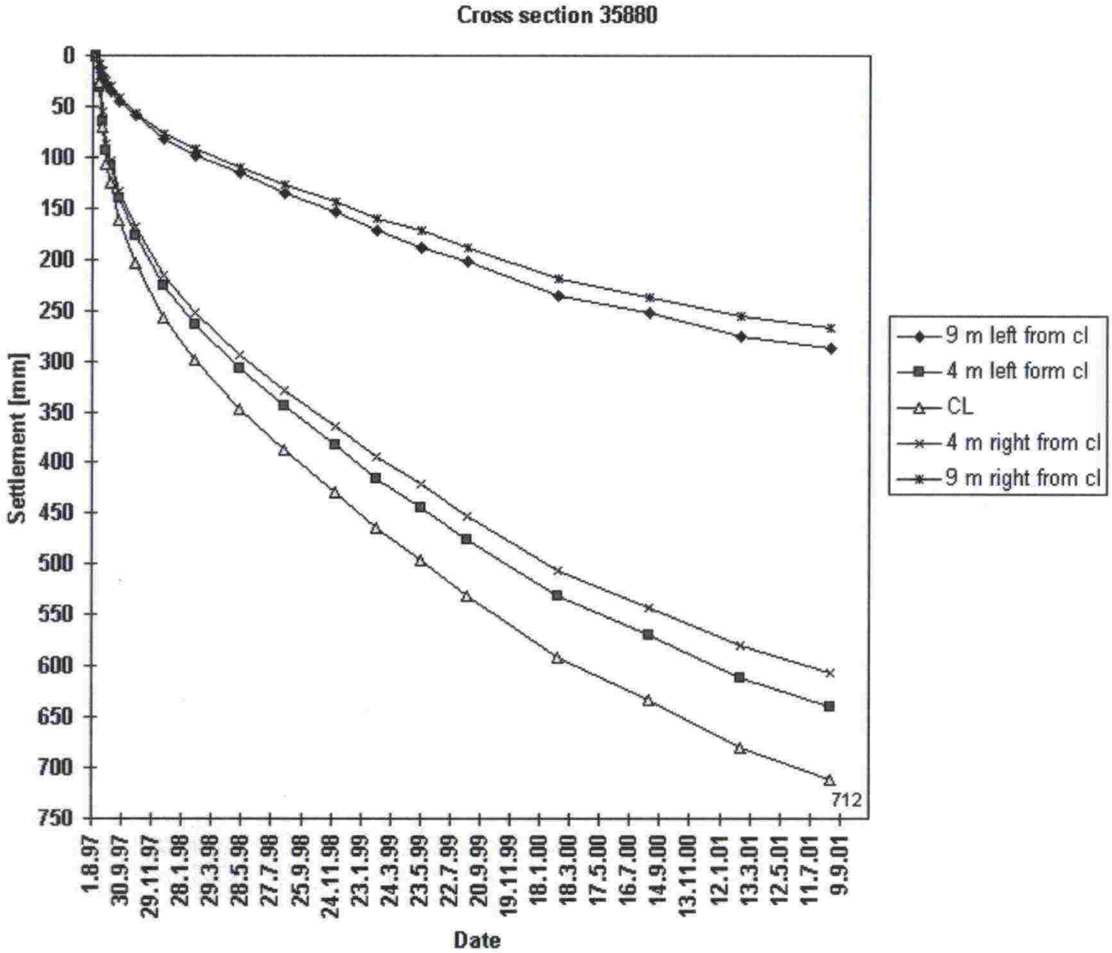

Figure 7.2. Settlement-time observations (upper curves natural subsoil, lower curvesdrained subsoil).

The results of the competition were very scattered. During the first phase of the competition the contestants were given the results of normal undisturbed samples and soil classification test results and odometer test results. It was thought that the first phase would be completed using conventional methods of calculation. During the second phase the competitors were given a large number of triaxial test results, this time also keeping in mind the possibility of using numeric methods. About 15 responses to the competition were submitted and the results are also posted on the Finnish Road Administration's Internet site.

Several different methods of calculation were used in the responses to the competition, from a conventional tangent modulus method to the most modern element method programs. The classic methods gave reasonably good settlement predictions, although some of the assumptions (thickness of the settling layer, state of consolidation) made by the competitors were not fully justified. The benefit of local experience was clearly observable.

Settlement calculation of the subsoil with vertical strip drains proved to be problematic, and the results of the calculations are not very easy to interpret, as primary settlement still continues after four years. The greatest predicted settlement after two years was about 2 meters. Reasonably accurate predictions of horizontal displacements and pore water pressure were also made using numeric methods.

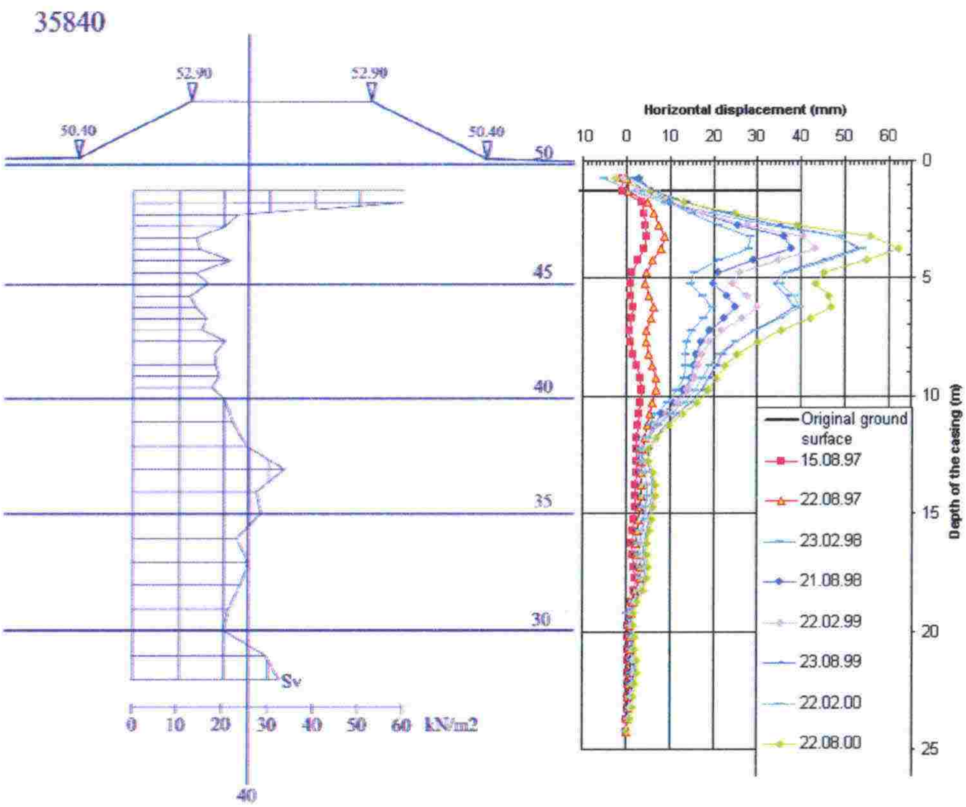

Figure 7.3. Cross section of the embankment and measured horizontral displacements

In a study /2/ conducted at Helsinki Technical University (HUT) after the competition, an attempt was made to explain the "strange" behavior of the embankment with vertical strip drains, but no unambiguous explanation could be found. The overconsolidated subsoil, in which the preconsolidation pressure is slightly exceeded in most layers, is one explaining factor. The Haarajoki clay is very sensitive, so the disturbance caused by the installation of the vertical strip drains may affect the rate of settlement.

The results of the Haarajoki test embankment settlement competition were modest, but they did provide many good experiences. The most significant long-lasting result of the whole calculation competition project was an excellent observation site and its measurement equipment. The project has also awakened much interest internationally. Queries have arrived from countries including the USA, Canada and France, where the Haarajoki material and results have been used in universities and colleges to test calculation applications and mechanical soil models.

The results will be used to test new methods of calculation. A similar test embankment was constructed in Murro in Seinäjoki in 1993. The 2 m high embankment has settled about 80 cm over an eight year period /3/. The results obtained from these test embankments are used in long-term mechanical modeling of soil and in research projects dealing with settlement calculation at HUT.

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7.2 Recycled materials in road structures

The use of recycled materials (industrial by-products) in road and earth constructions was studied and developed during Road Structures Research Programme (S4) implementation in the environmental geotechnology program headed and funded by Tekes (1994-99) /1, 2, 3, 4, 5/. The result was new structural designs and instructions on how to study materials to promote their use. Road construction was seen as the area of use with the most potential for recycled materials. Very usable materials include crushed concrete (Betoroc™), blast-furnace sand, fly ash from coal burning (with certain reservations), tyre chips and asphalt chips /6,7,3/.

Crushed concrete is manufactured by crushing the concrete waste of hollow slab factories and sorted waste from building demolition jobs. Crushed concrete (Betoroc™) is divided into four categories based on the raw material used. Each category has its own quality requirements (granularity, compression resistance, frost-susceptibility, material purity). To benefit its rebonding characteristics, on-site compaction of the crushed concrete must be done close to the optimal water content of the material. Crushed concrete is primarily used in the base coarse and sub-base coarse of roads, streets and other trafficed areas, where its good strength and load-bearing characteristics can be used to advantage, e.g., by decreasing the thickness of the underlying layers.

Granulated blast-furnace slag is a by-product of raw steel manufacturing that resembles natural sand in appearance. Granulated blast-furnace slag is a hydraulically (slowly) binding material. Due to the angularity of the grains, the material when compacted forms a strong structure. Granulated blast-furnace slag has better thermal characteristics than natural sand, and being a binding material, it has good load-bearing capacity as a structure. The use of bound blast-furnace slag as a load-distributing, thermal-insulating layer in pavements is established practice in Finland. Blast-furnace slag can also be used as a binder to stabilize old structural layers. Powdered blast-furnace slag (powdered slag) supplemented with a cement activator is used to strengthen clay and silt in deep stabilization and mass stabilization of soft soils

Fly ash from coal burning is a silt-like material composed of different-sized spherical particles and needle-like crystals. Fly ash is a strengthening material and its strengthening characteristics can be improved by adding activators (e.g., cement and lime). Fly ash's thermal conductivity is lower than that of natural aggregates.

The use of binder-stabilized fly ash is technically possible in all layers of the pavement where the amount of binder can be used to regulate the strength of the layer. Fly ash alone is mainly used in the (separating layer) base coarse and underlying structural layers. The varying quality of raw materials and different combustion processes cause variation in ash quality. For this reason the most important characteristics of the material that affect the success of the structure must be tested not only in preliminary tests, but also immediately before the construction phase. Special care must also be observed when examining the frost resistance and frost-heaving characteristics of ash /2/. Heavy metals in the ash and the risk of corrosion may also limit the use of ash in road and earth construction.

Tyre chips consist of tyre material chopped with a cutter. The products are categorized into three categories according to their chip size. The weight by volume of tyre chips is 2 – 6 kN/m^3. Tyre chips have a low thermal conductivity and they do not absorb water. Tyre chips can be used as a weight-reducing material in road, street and yard construction. Tyre chips are not suitable for use in a base or sub-base coarse. A layer of tyre chips is very springy, and the pavement must be thick enough to give the structure the required load-bearing capacity. Tyre chips or whole tyres can be used in noise barriers as a weight-reducing material and a fill material. Tyre chips are not hazardous to the environment.

Recycled asphalt means asphalt material formed when asphal pavements are milled and old pavements are removend and crushed. RC-asphalt resembles crushed aggregate (stone material content over 90 % by weight), and sometimes it is difficult to differentiate between RC-asphalt and pure crushed aggregate. The best technical benefit is obtained from RC-asphalt when it is recovered for asphalt stations and reused as a raw material of asphalt concrete. This method of use is sensible when the RC-asphalt is used as the aggregate material in the layers of pavements or other structures. Today recycled asphalts are included in the standard range of products of the asphalt industry.

The environmental geotechnology program included an extensive test construction program in which industrial by-products were used in different earth structures in a total of 21 test construction projects /3/. Many of the sites were roads, the Road Administration and Road Districts participated in financing and implementation.

The results obtained from the test construction sites have provided valuable additional information and new information on the use of recycled materials in earth structures. Based on the experience gained, the following prerequisites for the use of recycled materials are emphasized:

- the functional characteristics of the materials must be studied sufficiently well and sufficiently early before the material is used – the best way to promote usage is to create a product out of the material,
- the long-term behavior and durability of the material in a structure must be estimated on the basis of preliminary tests,
- the quality of the material has to be controlled throughout the construction period and possible changes in quality should be taken into consideration in the preliminary studies of the material (product creation),
- the environmental compatibility of the material must be studied using commonly accepted methods, and conclusions drawn on the basis of the studies must be presented unambiguously and adapted to the intended application,
- some by-products are very suitable for earth construction, but all industrial by-products are not usable materials in even slightly demanding earth structures.

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8 ROAD NETWORK LOADING, CONDITION AND FUTURE RESEARCH NEEDS

8.1 Stress caused by road traffic

Stress on the road network by traffic loading primarily consists of loading by heavy vehicles. The impact of passenger cars in Finland is mostly apparent as wear caused by studded tires, which has decreased significantly in recent years as less aggressive tyres and more wear-resistant pavements have become common.

Road transport volume increased robustly from the 1950s until the recession in the early 1990s (figure 8.1). Transport volume from 1960 to 2000 increased nearly fivefold. Heavy vehicle traffic volume only doubled during the same period. Transport volume in the 1990s grew only 5 %, while heavy vehicle traffic volume stayed the same.

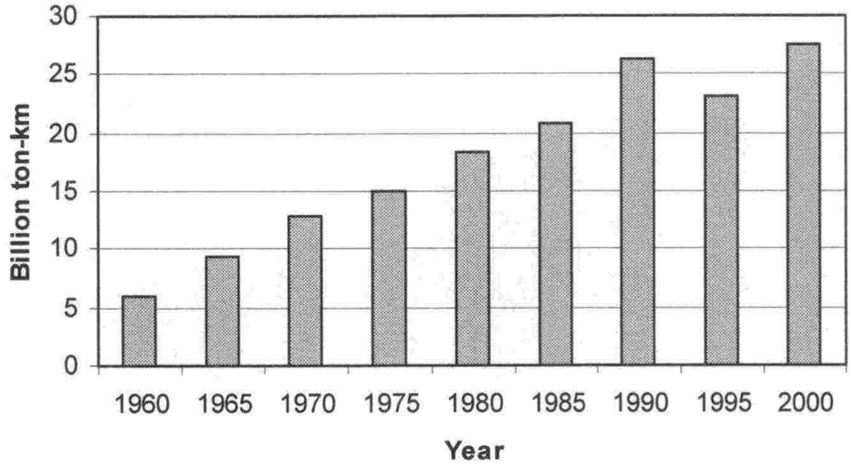

Figure 8.1. Finland's road transport volume, 1960-2000.

Goods transports are constantly increasing in size and employing heavier vehicles (table 8.1). Today more trailer trucks are used for transporting than before. At the same time the average total mass of the vehicles is increasing significantly.

Table 8.1. Distribution of truck traffic volume by vehicle type and average total mass in 1986 and 1999.

Vehicle type	Distribution of truck traffic, %		Total mass, ton	
	1986	1999	1986	1999
Trucks without trailer	40	25	11.0	12.9
Trucks with semitrailer	11	19	24.1	28.1
Trucks with full trailer	49	56	33.8	41.9
Average			23.7	32.1

In assessing stress it is necessary to consider not only total mass, but also axle and tyre loads. Despite the increase in the number of axles per vehicle, the mass of single and tandem axles has clearly grown (figure 8.2). The increase in maximum mass has been especially significant.

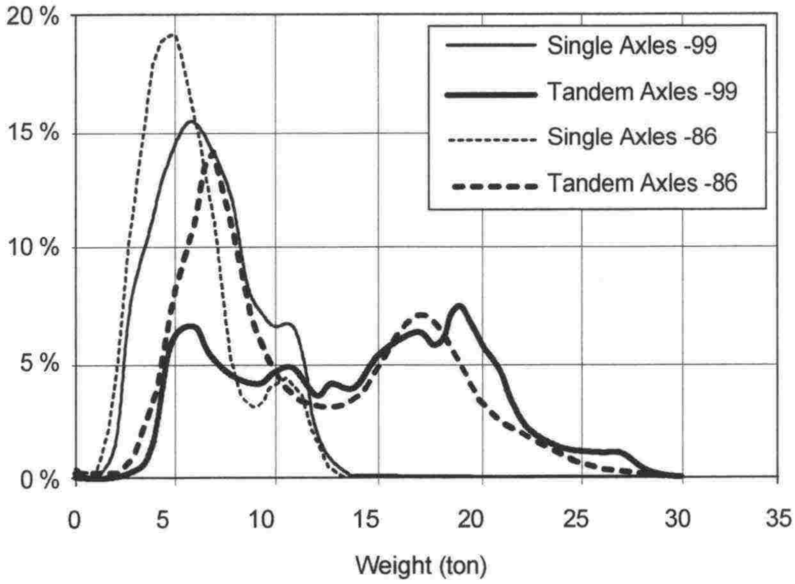

Figure 8.2. Distribution of single and tandem axle mass in 1986 and 1999.

It is becoming increasingly common to replace double tyres with single wide tyres (width > 350 mm), called supersingles. Supersingles cause more stress to the top of the road structure than double tyres. Tyre pressures are also higher and tyre structures are stiffer, which also increases stress.

Studies of axle masses in 1986 and 1999 specified the load equivalence of different vehicle types, which indicates how many 10-ton standard axles the stress caused by a vehicle type is equivalent to. Except for semi-trailer trucks, the load equivalence of vehicles increased during the period between the studies (table 8.2).

Table 8.2. Load equivalence of vehicle types in 1986 and 1999.

Vehicle type	Load equivalence	
	1986	1999
Trucks without trailer	0.4	0.6
Trucks with semitrailer	1.5	1.5
Trucks with full trailer	2.3	2.6

On the basis of changes in load equivalence and traffic volume of vehicle types, stress inflicted on the road network has increased about 20 % during the 1990s, although transport volume during the same period has remained nearly unchanged.

8.2 Condition of the road network

There were about 50,000 km of undivided paved roads in 2000. A little over 34 % of paved roads were asphalt concrete roads, 58 % were soft asphalt concrete roads and 7 % were surface dressing on gravel roads. The condition of the road network and its development during the 1990s was examined on the basis of a condition data register maintained by Finnra in terms of pavement damage and longitudinal and transversal unevenness.

The material is classified according to five different pavement types; AC1, AC2, SAC, VSAC and SD. Pavement type AC1 is a single-layer asphalt concrete. Pavement type AC2 contains several AC layers. Pavement type SAC is a soft asphalt concrete. Pavement type VSAC is a very soft asphalt concrete, which also includes old oil-gravel pavements. SD is surface dressing.

Depending on the type of pavement, in 2000 there were 300-360 m of longitudinal cracks per km (figure 8.3). The portion of the road network with longitudinal cracks was large, as nearly 60 % of 100 m sections of road contain some degree of longitudinal cracking. Longitudinal cracking on different pavement types has increased 15 %, on average, between 1994 and 2000. The amount of longitudinal cracks on AC1 roads increased as much as 40 %. Longitudinal cracking is primarily caused by frost heaving.

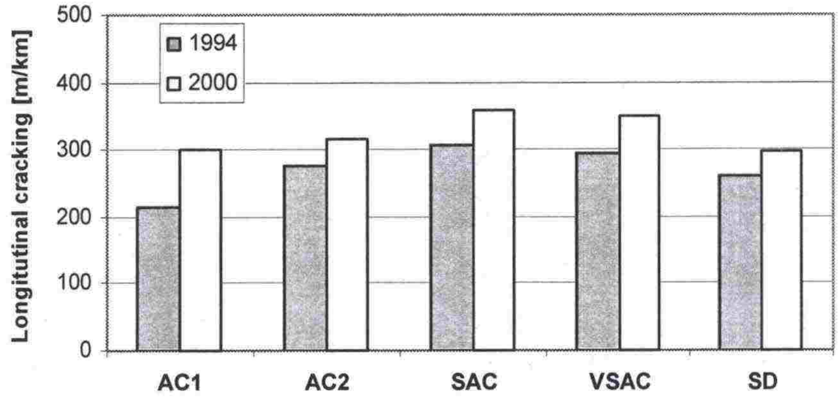

Figure 8.3. Amount of longitudinal cracking on different pavement types in 1994 and 2000.

The average number of transversal cracks was 10 per km in 2000. There were about 20 cracks per km on AC2 roads. The number of transversal cracks increased 1.5 to 2-fold between 1994 and 2000. Transversal cracks are primarily caused by shrinking of pavement at low temperatures, i.e., thermal tension.

The relative amount of alligator cracking on AC1 roads was noticeably higher than on AC2 roads in 2000 (figure 8.4). The amount of alligator cracking on VSAC and SD roads was clearly even greater than on other pavement types. No significant changes have happened in the amount of alligator cracking in six years.

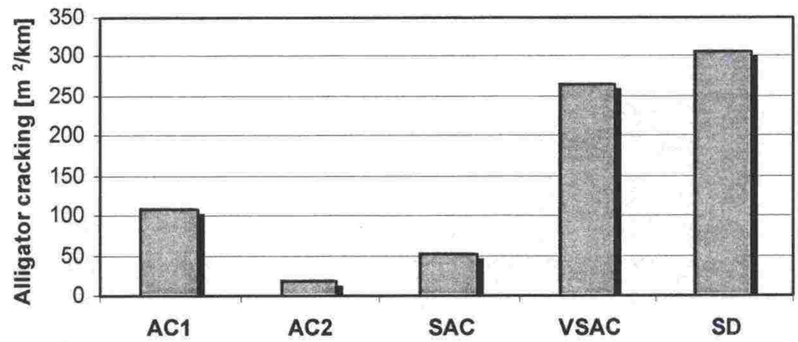

Figure 8.4. Alligator cracking per kilometer in 2000.

Formation of alligator cracking on thinly paved roads (≤ 80 mm) differs from fatigue damage of thickly paved roads. Permanent deformation (rut formation) in the unbound layers of thinly paved roads is considerable. As the bound layer adapts to the permanent deformation, transversal tension deformation appearing in the ruts may exceed the pavement's capacity to withstand movement. As a result the pavement may tear, forming longitudinal cracks in the middle of the rut. Consequently, stress caused by traffic loading increases, resulting in more damage, such as alligator cracking.

Longitudinal and transversal unevenness was noticeable only on part of the lower-class road network. The small amount of unevenness on the higher-class road network is due to the thick road structure and frequent repaving. Regardless of an increase in surface damage, longitudinal unevenness improved slightly, on average, between 1994 and 2000. Transversal unevenness on the poorest sections of the lower-class road network also decreased slightly, but rut depth on AC roads has increased.

The relatively light repairs commonly implemented in recent years have succeeded in keeping the paved road network in good condition in terms of longitudinal and transversal unevenness. On the other hand, surface damage, particularly longitudinal and transversal cracking, has increased in spite of yearly maintenance procedures.

The relatively light repairs currently implemented will not keep the condition of the paved road network at the present level over the long term. The condition of the road network will unavoidably get worse due to increasing longitudinal and transversal cracking and the resulting general deterioration of the road network's structural condition. If the condition of the paved road network is to be kept at the current level, maintenance and repair procedures need to be increased.

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8.3 Research and development needs

Although in recent years Finland has invested considerably in the research of road structures, there are still several issues that are not mastered. In changing to new contracting and procurement practices, the significance of understanding the behavior of road structures is emphasized even more from the client's and contractor's viewpoint. Good knowledge of the behavior of pavement would be very important, e.g., when designing structures containing new materials.

Today road construction in Finland concentrates strongly on improving the structure of existing roads rather than new roads. Stabilization using bitumen and hydraulic binders is well suited to structural improvement. Despite this, Finland currently does not have a method that is well suited for designing stabilized structures. For this reason, clearly incorrect solutions have also been implemented.

The characteristics of the old bound layer, such as stiffness, have a significant effect on the performance of a repaved road structure. Currently there is no method with which damage to the old bound layer and the significance of the damage to the improved road structure can be taken into consideration.

In Finland, moisture and moisture variation have a very large effect on the performance and damage of road structures. Regardless of this, moisture variation, factors affecting moisture and the impact of moisture on structural performance are understood very poorly.

Especially on the lower-class road network, permanent deformation of the unbound layers and subgrade are crucial from the standpoint of the performance of the whole structure. In spite of this, consideration given to permanent deformation is still at an elementary level.

Traffic volume and composition and the surface condition of the road network are monitored regularly. But, actual stress caused by traffic loading is not measured, and the structural condition of the road network is monitored very little. Continuous monitoring of stress would be possible with today's measurement and information technology methods. By measuring stress and condition information in different types of road structures, it would be possible to create extensive data with which to determine the impact of various types of loading and conditions on the performance and damage of different road structures.

A continuously operating system for measuring stress and environment factors has been tested in Finland since 1997. The system and obtained results have proved to be reliable. A system for monitoring the structural condition of a road and prevalent environment factors could be combined with an existing automatic traffic monitoring system. Then it would be possible to obtain information on both traffic and the behavior of the road structure.

A life cycle cost calculation procedure developed during the TPPT project was partly incomplete and inaccurate. Because of the inaccuracy and incomplete development of the road performance prediction models (life span models), differences between different structural solutions could not be detected sufficiently well. Among the reasons for the inaccuracy of the models are the non-homogeneity of the subsoil and construction materials, variation in the quality of construction, the complex behavior of the road structure and the non-homogeneity of old road structures.

It is absolutely necessary to increase the accuracy of pavement performance models so that procedures used in the construction and maintenance of road structures can be scheduled and designed overall-economically. New possibilities are offered by more accurate road network data handling, where the non-homogeneity of the road network can be controlled and road construction and maintenance procedures can be more accurately recorded into data registers.

1. ROAD STRUCTURES RESEARCH PROJECT (TPPT) REPORTS

Most of the TPPT reprts can be read and printed from Internet (pdf-format) http://www.tiehallinto.fi/tppt/s4_julkaisut.htm, or via www.finnra.fi/tppt
The reports published in Finnish have an English abstract.

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Tiehallinto 66/2001	Alkio, R., Juvankoski, M., Korkiala-Tanttu, L., Laaksonen, R., Laukkanen, K., Petäjä, S., Pihlajamäki, J. & Spoof, H.	Tien rakennekerrosten materiaalit. Taustatietoa materiaalivalinnoille. *Materials in the pavement structural layers. Background information for material selection*
Tiehallinto 8/2002	Kivikoski, H. & Pihlajamäki, J. & Tammirinne, M.	TPPT- koerakenteiden yhteenvetoraportti. *TPPT test construction sites. Results*

Invstigation methods descriptions (all in Finnish)

TPPT 1	Spoof, H. & Petäjä, S.	Pudotuspainolaitemittaus (PPL-mittaus) *Falling weight measurement*
TPPT 2	Spoof, H. & Petäjä, S.	Rakennekerrosmoduulien takaisinlaskenta sekä jännitysten ja muodonmuutosten laskenta *Back-calculation of structural layer modules and calculation of stress and deformation*
TPPT 3	Pihlajamäki, J.	Liikennerasituksen laskeminen *Calculation of traffic loading*
TPPT 4	Kivikoski, H. & Saarelainen, S.	Ilmastorasitus. Pakkasmäärän ja sulamiskauden pituuden määritys *Climatic stress. Determining the freezing index and the length of the thawing period*
TPPT 5	Onninen, H.	Roudan syvyyden määritys *Determining the depth of frost penetration*
TPPT 6	Onninen, H.	Routanousukoe. Routimiskertoimen (SP) kokeellinen määritys Frost heaving test. *Experimental determination of the frost heave coefficient (SP)*
TPPT 7	Saarelainen, S.	Routimiskertoimen määritys *Determination of the frost heave coefficient*
TPPT 8	Kivikoski, H, Saarelainen, S., Ahonen, M., Huttunen, E. & Kujala, K.	Lämmönjohtavuuden määrittäminen *Determination of the thermal conductivity*

TPPT 9	Törnqvist, J., Laaksonen, R. & Juvankoski, M.	Sähköinen vastusluotaus tien painumalaskennan lähtötietojen hankkimisessa *Using electrical resistance sounding to obtain data for road settlement profile calculation*
TPPT 10	Törnqvist, J. & Laaksonen, R.	Radiometrinen reikämittaus *Radiometric hole measurement*
TPPT 11	Törnqvist, J. & Tammirinne, M.	CPTU - kairaus *CPTU test*
TPPT 12	Törnqvist, J., Laaksonen, R. & Juvankoski, M.	Läpäisevän kerroksen määrittäminen painumalaskennan tarpeisiin *Determination the permeable layer for settlement profile calculation*
TPPT 13	Korkiala-Tanttu, L. & Onninen, H.	Tien rakennekerrostutkimukset *Investigation of the structural layers of a road*
TPPT 14	Onninen, H.	Routanousun ja painuman mittaus *Measuring frost heave and settlement*
TPPT 15	Onninen, H. & Spoof. H.	Tien vauriokartoitus ja vaurioiden kuvaus *Road damage survey*
TPPT 16	Onninen, H.	Palvelutasomittaus (PTM) tien rakenteen parantamisen suunnittelussa *Pavement surface measurements in designing road structure improvement*
TPPT 17	Spoof, H. & Pihlajamäki, J.	Kuormituskestävyysmitoitus. Päällysrakenteen väsyminen *Fatigue design of the pavement structures*
TPPT 18	Saarelainen, S.	Tierakenteen routamitoitus *Frost design of a road structure*
TPPT 19	Törnqvist, J., Laaksonen, R., Juvankoski, M., Vepsäläinen, P., Lojander, M. & Takala, J.	Tien jatkuvan painumaprofiilin laskenta pixelimallilla *Pixel model to calculate the continuous settlement profile of a road*
TPPT 20	Petäjä, S. & Spoof. H.	Päällysrakenteen elinkaarikustannusanalyysi *Life cycle cost analysis of the pavement*
TPPT 21	Onninen, H.	Tierakenteen mitoituksen lähtötietojen hankkiminen *Investigation data for road structure design*

Reports (all in Finnish)

TPPT 22	Juvankoski, M. & Laaksonen, R.	Sitomattomat tien rakennekerrosmateriaalit. Taustatietoa materiaalien käyttäytymisestä. *Unbound materials in the pavement structures. Background information about material behavior.*
TPPT 23	Saarelainen, S.	Pohjamaan urautumisen ja sulamisen arviointi kevätkantavuusvaiheessa *Estimation of subsoil rut formation and thawing during the springtime load-bearing phase*
TPPT 24	Törnqvist, J. & Juvankoski, M.	Riski ja luotettavuus tierakenteiden suunnittelussa *Risk and reliability in road structure design*
TPPT 44	Juvankoski, M., Laaksonen, R. & Törnqvist, J.	Pohjamaan moduuli ja sen määrittäminen CPTU-kairauksella *Subsoil module and its determination based on CPTU tests*
TPPT 45	Gustavsson, H. & Saarelainen, S.	Routanousuvaurioriskin arviointi *Estimation of the risk of damage due to frost heave*

Test structure reports (all in Finnish)

TPPT 25	Alkio, R. & Pihlajamäki, J.	Kehä III. SMA + ABS + ABK, Maabetoni + lujite *SMA + ABS + ABK, Soil cement + reinforcement*
TPPT 26	Leinonen, S., Sikiö, J. & Pihlajamäki. J.	Kehä II. AB (B-200)+ABS (Gilsonite)+SMA *AB (B-200)+ABS (Gilsonite)+SMA*
TPPT 27	Alkio, R.	Jutikkalan eritasoliittymä. Paksu bituminen sidottu ABK (B-80), ABK (B-25). Komposiittirakenne, Bitumistabilointirakenne, Maabetonirakenne *Thick bitumen bound ABK (B-80), ABK (B-25). Composite structure, bitumen stabilized structure, soil cement structure*
TPPT 28	Laukkanen, K., Pienimäki, M., Pihlajamäki, J. & Sikiö, J.	PT 12895 Nakkila. Komposiittirakenne *Composite structure*
TPPT 29	Juvankoski, M. & Kivikoski, H.	Mt 272 Ämttöö. Lento- ja pohjatuhka tien päällysrakenteessa *Fly ash and bottom ash in the base course of a road*
TPPT 30	Laukkanen, K., Pienimäki, M. & Pihlajamäki, J.	Vt 19 Seinäjoki. Komposiittirakenne *Composite structure*
TPPT 31	Apilo, L. & Pihlajamäki	Mt 661 Isojoki. Bitumistabilointi, Raudoiteverkko *Bitumen stabilization, reinforcement mesh*
TPPT 32	Kivikoski, H. & Pihlajamäki, J.	Mt 718 Vöyri. Bitumistabiloitu päällysrakenne *Bitumen stabilized superstructure*
TPPT 33	Alkio, R. & Pihlajamäki, J.	Vt 5 Juva. Sitomattoman murskeen koerakenteet *Unbound crushed aggregate structures*
TPPT 34	Alkio, R.	Vt 4 Leivonmäki. Levennykset, lujitteiden käyttö *Widening the road, use of reinforcement*
TPPT 35	Ahonen, M., Holappa, T., Huttunen, E., Kivikoski, H.	Mt 595 Kiuruvesi. Alusrakenteen homogenisointi+stabilointi *Homogenization + stabilization of the substructure*
TPPT 36	Kivikoski, H.	Mt 5950 Salahmi. Alusrakenteen homogenisointi+stabilointi *Homogenization + stabilization of the substructure*
TPPT 37	Ahonen, M., Holappa, T., Huttunen, E. & Kivikoski, H.	Pt 18564 Rantsila. Turverakenne ja masuunihiekkarakenne *Peat structure and blast furnace sand structure*
TPPT 38	Ahonen, M., Holappa, T., Huttunen, E. & Kivikoski, H.	Pt 18629 Temmes. Raudoiteverkko, *Reinforcement mesh*
TPPT 39	Ahonen, M., Holappa, T., Huttunen, E. & Kivikoski, H.	Vt 4 pyörätie Temmes Turverakenne, LD-teräskuonarakenne, Geoprofiilirakenne, Raudoiteverkko *Peat structure, LD steel slag structure, geoprofile structure, reinforcement mesh*
TPPT 40	Ahonen, M., Holappa, T., Huttunen, E. & Kivikoski, H.	Kt 83 Pello. Turverakenne, Massanvaihto *Peat structure, mass replacement*
TPPT 41	Ahonen, M., Holappa, T., Huttunen, E. & Kivikoski, H.	Kt 78 Ranua. Turverakenne, Raudoiteverkko *Peat structure, reinforcement mesh*

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